FLOOD ESTIMATION IN SMALL CATCHMENTS —A REGIONAL CASE STUDY

A Thesis Submitted
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by

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to the

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MAY, 1992

to

my daughters

Shruti, Shraddha and Shreya

[illegible]

CE-1992-D-JAY-FLO

CERTIFICATE

It is certified that the work contained in the thesis entitled "FLOOD ESTIMATION IN SMALL CATCHMENTS - A REGIONAL CASE STUDY" *by K V Jayakumar has been carried out under my supervison and that this work has not been submitted elsewhere for a degree.*

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K V Jayakumar

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LIST OF SYMBOLS AND ABBREVIATIONS

A	Basin area
ARI	Average recurrence interval
a,b,c,d	Regression coefficients
BMS	Base moisture storage
BMSN	Soil moisture storage at field capacity
C	Constant
C_v	Coefficient of Variation
C_s	Coefficient of skewness
CAZRI	Central Arid Zone Research Institute
CSWCRTI	Central Soil and Water Conservation Research and Training Institute
Cusecs	Cubic feet per second
Cumecs	Cubic meters per second
CWC	Central Water Commission
DDF	Depth- duration- frequncy
DR3M	Distributed Routing Rainfall Runoff Model
DRH	Direct runoff hydrograph
DSRO	Direct surface runoff
ERH	Effective Rainfall Hyetograph
H	Stage
H_0	Hypothetical datum corresponding to zero discharge
I	Intensity of rainfall
ICRISAT	International Crop Research Institute for Semi-Arid Tropics
IMD	India Meteorology Department
IUH	Instantaneous unit hydrograph
K	Nash model parameter
KSAT	Effective saturated value of hydraulic conductivity
L	Length of longest main stream along the river course
L_c	Length of the longest main stream from a point opposite to centroid of the catchment area to the gauging site along the main stream
m_0	Uniformly distributed moisture content of soil column, when infiltration started
m	Moisture content, uniformly dsitributed through wetted column at the time at which infiltration rate is being computed

n	constant
N	Nash model parameter
P_v	Average precipitation over the catchment
PS	Capillary potential at wetting front
PSP	Suction at wetted front for soil moisture at field capacity
q_b	Baseflow per unit area
q_p	Peak discharge of unit hydrograph/unit area
Q	Discharge rate
Q_m	Maximum flood
Q_p, Q_{peak}	Peak discharge rate
R_e	Rainfall excess
RDSO	Research Designs and Standards Organisation
RGF	Ratio of suction at wetting front for soil moisture at wilting point to that at field capacity
RR	Proportion of daily rainfall that infiltrates into the soil for the period of simulation excluding unit rainfall days
S	Storage, stream slope
SMS	Saturated moisture storage
SR	Supply rate of rainfall for infiltration
t_B	Base length of UH
t_c	Time of concentration
t_p	Time of rise of UH
UH	Unit hydrograph
V	Volume of runoff
W_{50}	Width of UH measured at 50% peak discharge ordinate
W_{75}	Width of UH measured at 75% peak dischrge ordinate
W_{R50}	Width of riding side of UH measured at 50% of peak discharge ordinate
W_{R75}	Width of riding side of UH measured at 75% of peak discharge ordinate
$X_1, X_2 \ldots$	Factors controlling flood peaks
Z	Fraction of unitgraph peak for a duration t to the equilibrium discharge
α, m	Kinematic wave parameters

SYNOPSIS

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FLOOD ESTIMATION IN SMALL CATCHMENTS - A REGIONAL CASE STUDY

Summary:

Estimates of flood are required for the design and economic appraisal of a variety of engineering works such as culverts, spillways etc. When adequate and appropriate data are available for the basin, frequency analysis may be applied or else unit hydrograph (UH) and other rainfall-runoff procedures may be applied to model the basin system. When this is combined with design rainfall, the design flood may be derived. Data available for small watersheds are generally inadequate for these approaches. Regional flood frequency analysis is generally not valid for small watersheds because of data limitations. Then, other empirical approaches for regionalisation are to be used to model either the variation of flood peaks or rainfall-runoff relationships.

From available data, regression relationships are derived to relate peak discharge to hydrometeorological features of the storm and physiographic features of the watershed. Even when adequate data are available for establishing such relationships, they are limited to the regions for which such relationships have been derived. Since generally adequate data are available for estimating design storms, it seems worthwhile to derive relationships between storm rainfall and streamflow. The parameters of the model may then be related to the characteristics of the storm and the physiographic features of the watershed to yield a regional relationship between storm and floods.

A more realistic and alternate approach is to model the catchment runoff process according to physical principles. Information regarding physiography, soils, vegetation and the drainage pattern of the catchment is used in the physically based models. The catchment is decomposed into a network of channels and overland flow segments. Using the physical characteristics of each component and appropriate hydraulic equations, the hydrologic processes are simulated using the distributed parameter model. Necessary values of the coefficients and constants are specified from physical characteristics of the watershed or obtained from calibration of the model with observed data.

This study is essentially a regional case study for flood estimation in small catchments in a part of the North Brahmaputra basin, designated as subzone 2(a) by the Central Water Commission (CWC), to compare in a very limited data environment the two approaches for rainfall-runoff relationships, viz., regionalisation using UH; and a physically based model .

Objectives of the Study: The objectives of the present study are:

i) To develop a regional relationship for UH parameters from available limited data for a number of basins to evaluate the validity and applicability of UH based regionalisation procedure for flood estimation;

ii) To apply a physically based model to small watersheds in the region; study the regional variation, if any, of the parameters of the model and also to judge their validity for flood estimation; and

iii) To compare these two approaches for regional flood estimation for small catchments.

Scope of the Study: The study is limited to small watersheds in the western part of the North Brahmaputra basin, for which data were availble. Furthermore,

the UH is represented by the Nash conceptual model and the physically based model considered in the study is the USGS Distributed Routing Rainfall Runoff Model (DR3M).

Data Used: Catchment plan, crosssection at the bridge site and hourly rainfall and stage data at railway bridges for about 10 catchments in the region were available for the study. The catchment areas ranged from about 7.00 Km^2 to 215.00 Km^2. The data were available for a period ranging from two to three years. After preliminary analysis for data errors, only data of seven basins were found to be suitable for analysis.

Details of the Study:

In the UH approach for regionalisation, it is necessary to represent the UH in terms of a limited number of parameters. Nash model with two parameters N and K was adopted in the study.

Attempts to correlate coefficient of runoff, the infiltration index and base flow with storm and runoff parameters were unsuccessful. Variation of the infiltration index among the storms was very large. However, a value of 5.00 mm/hour can be considered to be the general minimum value for the infiltration index for all the basins, particularly during intense storms generally met with in the design. This value corresponds to the silty loam soil or sandy clay loam soil of the region.

The various steps involved in the regionalisation of the parameters of this model are:

i) Estimate the UH parameters for a storm in a basin;

ii) Repeat the step for a number of storms in a basin to explain the variation, if any, of the parameters in terms of hydrometeorological characteristics of the

storm; and

iii) Repeat the above two steps for a number of basins in the region so that the residual variation of parameters can be correlated to the physiographic characteristics of the basin.

The UH parameters N and K of the Nash model vary from storm to storm and from basin to basin. A smaller NK generally leads to a quicker flood and a high peak. A quasilinear variation of N and K as a function of peak discharge (Q_{peak}) is assumed and for each of the basin, the regression equations relating N, K and NK with Q_{peak} are derived. Regional regression equations relating N and K corresponding to the maximum observed peak in each basin is as follows:

$N = 1.792 + 0.0088\,A$, and

$K = 2.766 + 0.016A$

where A is in Km^2, K is in hours and N is dimensionless.

Baseflow varies from storm to storm and generally increases with the basin area. Since the higher values of baseflow lead to larger floods, the enveloping curve for baseflow/Km^2 as a function of basin area is recommended for use with the design flood.

A design flood estimated using the regional relationships for a small watershed in the region compares with other approaches generally used in India satisfactorily.

A drainage basin is represented in the USGS DR3M as a set of channel and overland flow segments in such a way that the essential basin geometry and physiography are taken into account for runoff computation. The model has four components - a soil moisture or water balance component, the infiltration or

rainfall excess component, a routing component and an optimisation component. The model has seven physically based parameters - four soil moisture accounting parameters viz., (i) DRN, a constant drainage rate for redistribution of soil moisture in inches/day, (ii) EVC, a pan coefficient for converting measured pan evaporation to potential evapotranspiration; (iii) RR, the proportion of daily rainfall that infiltrates into the soil for period of simulation excluding unit rainfall days and (iv) Soil-moisture storage at field capacity in inches, and three infiltration parameters viz., (i) PSP, the suction at wetted front for soil moisture at field capacity in inches of pressure, (ii) KSAT, the effective saturated value of the hydraulic conductivity in inches/hour and (iii) RGF, the ratio of suction at wetted front for soil moisture at wilting point to that at field capacity. These parameters are fitted to the model for a basin by Rosenbrock's algorithm and the iterative optimisation among basins is done heuristically.

It is realised from the study that the directly contributing impervious area percentage to be given as input data from the characteristics of the watershed is very important in estimating the flood peak. Such data were not available for the region. They were parametrised and estimated by simulation and compared with general topography and basin area and are considered to be satisfactory. However, with the use of remote sensed data (not available for the study but generally available for government organisations), it will be possible to use this information for realistic modelling of small watersheds. The results of the study indicated that a constant parameter model for the small catchements in the region is realistic. The model parameters viz., PSP = 10 inches; KSAT = 0.178 inches/hour; RGF = 18; and BMSN = 10 inches agree with the watershed characteristics. The design flood estimated for a small watershed using this model compared favourably with other

estimates.

Conclusions:

1. Since design rainfall can be estimated separately by standard procedures, it seems preferable to use for estimation of design flood in a small basin, a regional rainfall-runoff relationship with design storm rather than using a purely empirical approach.

2. For the design flood for the region under study, a constant inflitration parameter during the design storm, a baseflow related to the basin area and a regional quasilinear relationship for the UH parameters were found to be satisfactory. Regional UH approaches involve a large amount of empiricism. When sufficiently large amount of data are available they may be used for regionalisation.

3. DR3M is a relatively simple distributed parameter model which can be used for simulation of runoff in small watersheds. Impervious area is a very important parameter in the DR3M which has to be estimated with care.

4. Even in a very limited data environment, DR3M has been found to work satisfactorily.

5. Since field data concerning topography, soil and vegetation may be easily collected by remote sensing and other means, DR3M may be used when data are very limited.

6. Because of easy availability of, and competence with computers and softwares in field organisations, it seems preferable to adopt a distributed parameter model with relevant field data rather than empirical, statistical relationships of questionable validity.

7. Distributed parameter models, may, eventually repalce empirical regression models in hydrology not only for large watersheds but also for small watersheds.

CHAPTER 1

INTRODUCTION

1.1 General

A flood is any large peak flow, but the term is generally used to mean a flow that exceeds the capacity of the banks of the stream at a given location, thus creating potential damage. When a stream is *in flood* at one location, it does not mean that it is *in flood* all along its length. The estimation of flood peak is location specific. It is very important for design of various water control structures. A very major problem in hydrology is the prediction of magnitude of flood peaks at a site for design purposes.

The primary requirement for any such hydrologic analysis and design is the availability of data, especially those pertaining to rainfall and runoff in streams. Larger catchments, in general, are gauged at various locations. But for smaller catchments, the major constraint is the nonavailability of data in the stream at or near the site. A small catchment may be defined as one that is so small that its sensitiveness to high intensity rainfalls of short durations and to land use are not suppressed by the channel storage characteristics (Chow,1964). Numerous attempts have been made to delimit the *small* catchment either by area; or as function or type of storage.

Only a limited number of small watersheds are gauged throughout the country and a few studies have been carried out on hydrometeorological aspects of small watersheds. Problem specific studies and data collection have been carried out by research organisations in India like International Crop Research Institute for the Semi-Arid Tropics (ICRISAT), Central Arid Zone Research Institute (CAZRI), Central Soil and Water Conservation Research and Training

Institute (CSWCRTI) and Research Designs and Standards Organisation (RDSO) of the Ministry of Railways. One of the primary reasons for the limited number of general studies has been the lack of data. In the absence of data, the engineer has to find ways and means to estimate flood peaks. Since small structures are associated with small watersheds, it is possible to accept some risk of failure of these small structures and so they are generally designed for a small probability of failure, say, in terms of recurrence interval of storms or floods for design. Estimation of floods for various recurrence intervals for watersheds where no observations has been made has been a challenging task for engineers who are involved in hydrologic analysis and design.

Several approaches are available for estimation of floods. When adequate and appropriate data are available for the basin, frequency analysis may be used or else unit hydrograph (UH) and other rainfall-runoff procedures may be applied to model the basin system. When this is combined with design rainfall, the design flood may be derived.

Data available for small watersheds are generally inadequate for these approaches. In such a case, empirical approaches are to be used to estimate the design flood. This can be through the development of a relationship either for the regional variation of flood peaks or for rainfall-runoff relationships. From available data, regression relationships are derived to relate peak discharges to hydrometeorological features of the storm and physiographic characteristics of the basin. Even when adequate data are available for establishing such relationships, they are limited to the regions for which such relationships have been derived.

Regional frequency analysis is generally not possible for small watersheds

because of data length limitation. Since generally adequate data are available for estimating design storms, it seems worthwhile to derive relationships between storm rainfall and streamflow, perhaps in terms of unit hydrographs. The parameters of the UH may then be related to the characteristics of the storm and physiographic features of the watershed to yield a regional relationship between storm and flood.

A more realistic approach is to simulate the catchment runoff process according to physical principles. Data concerning topography, soil and vegetation can be obtained from remote sensing. The physically based distributed parameter models can now be implemented even in a PC- environment. There has been also a greater availability of technical know-how on the applicability of such models. They may also eliminate the need for regionalisation. Hence it seems reasonable to consider the use of physically based distributed parameter models as an alternative to regionalisation for flood estimation in small catchments. Thus the study proposes to explore the possibility of using physically based distributed parameter models for flood estimation and to compare them with a regionalisation procedure based on UH approaches.

1.2 Statement of the Problem

Small catchments are characterised by:

i) relatively short duration of storm,

ii) a nearly uniform rainfall distribution in space and time,

iii) steeper slopes and relatively less permeable surfaces,

iv) predominance of overland flow and channel flow characteristics in comparison to storage characteristics,

v) a nearly wet condition at the beginning of design storms, and

vi) generally a quick response of the catchment in producing runoff.

Empirical relationships are often region specific and may not be available for the region under study; and small watershed data are generally of very short length thus precluding regional frequency approaches. Design storms for storm precipitation can be fairly well estimated from the data available with India Meteorology Department (IMD). Accordingly estimation of design flood in a small watershed reduces to establishing the rainfall-runoff relationships for the watershed and combining with design rainfall to produce design runoff including peak runoff.

Rainfall-runoff relationships for small watersheds can be regionalised as follows:

i) Estimate the UH parameters for a storm in a basin,

ii) Repeat the step for a number of storms in a basin to explain the variation, if any, of the parameters in terms of hydrometeorological characteristics of the storm; and

iii) Repeat the above two steps for a number of basins in the region so that the residual variation of parameters can be correlated to the physiographic characteristics of the basin.

Alternatively a distributed parameter model may be fitted to the data from each watershed. Information regarding physiography, soils, vegetation and the drainage pattern of the catchment is used in the physically based models. The catchment is decomposed into a network of channel and overland flow segments. Using the physical characteristics of each component and appropriate hydraulic equations, the hydrologic processes are simulated using the distributed parameter model. Necessary values of the coefficients and constants are specified from physical characteristics of the watershed or obtained from calibration of the model

with observed data. In case the value of any of the parameters vary very significantly, then regional variation may be modelled appropriately. While such a variation may be large in large basin and in different regions, they are expected to be fairly small for small watersheds in a region. A comparison of the performance of the two approaches in modelling the rainfall-runoff relationships in small watersheds may perhaps lead to identification of better procedures.

For purposes of hydrometeorological studies, India has been divided into 26 subzones by the Central Water Commission (CWC), each of which is assumed to be hydrologically homogeneous. Research Designs and Standards Organisation (RDSO) of the Ministry of Railways has collected and in collaboration with CWC and IMD has analysed a large amount of hydrometeorological data for a large number of catchments draining into various railway culverts and bridges in different subzones. However they seem to have problems in the analysis of data and regionalisation in flood estimation for small watersheds because of large errors. Hence the estimation of floods in small watersheds, particularly when there are serious data limitations, is very important. Thus it seems worthwhile to consider the suitability of the models in a data scarce environment.

1.3 Objectives of the Study

This study is essentially a regional case study for flood estimation in small catchments in a part of the North Brahmaputra basin with a very limited data environment to compare the two approaches for rainfall-runoff relationships, viz., regionalisation using unit hydrograph (UH) approach and a physically based distributed parameter model. The objectives of the study are:

i) To develop a regional relationship for UH parameters from available limited

data for a number of basins to evaluate the validity and applicability of UH based regionalisation procedures for flood estimation;

ii) To apply a physically based model to small catchments in the region, study the regional variation, if any, of the parameters and also to judge their validity for flood estimation; and

iii) To compare the two approaches for regional flood estimation for small catchments.

1.4 Significance of the Study

Development of regional procedures for estimation of design floods in small to medium sized drainage basins is an important problem in hydrology. Even though many methods have been developed and used widely, in many cases, there is a serious doubt about their adequacy and suitability.

Small watershed hydrology has received limited attention in research and design because of a number of factors, viz.,

i) Small watersheds have small hydraulic structures and projects with low expenditure or financial investment;

ii) A routine design carried out in a minimum time is considered to be satisfactory;

iii) Unfortunately the large number of such structures or projects is not taken note of, and hence economy and scientific design in this area are ignored; and

iv) This is not considered to be an intellectually or scientifically satisfying area of study for researchers (Pilgrim,1989).

Yet this area is becoming very important because of the potential saving in

the very large number of hydraulic structures and projects in the small watersheds. Any significant improvement for rational hydrologic design will lead to technically viable and economical system. Such procedures have been developed and applied in urban drainage systems which are relatively small. Accordingly, the study objectives are considered to be very important for rational scientific design with a better insight into the hydrology of small watersheds and hence the study is considered to be significant.

1.5 Scope of the Study

The scope of the study is limited because of a number of factors:

i) Because of data availability relevant to the problem under consideration, the study is limited to a part of the North Brahmaputra basin;

ii) While several models are available for establishing rainfall-runoff relationship using a black box approach, the study is limited to UH approach using Nash Model and a combination of optimisation and heuristics in parameter estimation for the model;

iii) Similarly while several distributed models for a watershed are available, this study is limited to the USGS DR3M in case of physically based distributed parameter model; and

iv) Some of the details of the analysis are limited by the inaccessibility and remoteness of the region for collection of additional data, and in the final stages due to nonavailability of remote sensed data in limited time.

1.6 Organisation of the Thesis

The study is reported in the following sequence:

i) Flood estimation procedures in small watersheds with particular reference to the study are briefly reviewed in Chapter 2;

ii) Data availability, their limitations, preliminary analysis and selection of models used in the study are reported in Chapter 3;

iii) Chapter 4 contains the unit hydrograph approach using the Nash Model used for regionalisation. The analysis of the various basins used in the study are also presented therein;

iv) Chapter 5 contains a brief description of the USGS Distributed Routing Rainfall Runoff Model and its application to the basins chosen for the study;

v) The last chapter (Chapter 6) presents the summary of the work carried out, the conclusions and suggestions for future work.

C H A P T E R 2

FLOOD ESTIMATION PROCEDURES IN SMALL CATCHMENTS - A REVIEW

2.1 Introduction

Lack of adequate runoff information is one of the major problems faced world over by hydrologists and engineers who are involved in the design of water control structures and in the evaluation of surface water potential. Engineers are required to estimate the peak discharges at locations where there are adequate or dearth of data. The term ***dearth*** may mean nil or it may mean inadequate. When there are no data or when the available data are very limited, the hydrologist has to find ways and means of somehow rationally estimating the design flood. In such cases, it becomes necessary to interpolate or extrapolate the available data for the basin under study in time and over events, or use the information from nearby sites in the basin or from neighbouring streams and basins. These approaches are used to establish spatial correlations or regional relationships, as the case may be, for flood estimation. If the regional approach is used, considerable care should be taken to select streams as nearly similar in hydrologic characteristics as possible. They may have similar vegetal cover, land use, topographic conditions and geologic characteristics and the range of extrapolation of drainage area should not be excessive. They should also have similar rainfall and evapotranspiration regimes. The data pertaining to hydrology, topography, drainage network, soils and vegetation from several locations within a region are collected and analysed to derive empirical relationships for flood estimation for ungauged sites within each region. The procedures for data analysis and flood estimation may depend upon the length of record, the nature of variables, their accuracy and reliability and the amount of data available.

2.2 Factors Controlling Runoff

The factors that control the magnitude of runoff in a region can be broadly classified into permanent factors and transient factors (Fig. 2.1). The permanent factors include the basin characteristics such as the size, shape, slope etc.; drainage network characteristics like density, order, length of channel etc.; and channel characteristics such as crosssectional area, roughness and slope. The transient factors consist of storm characteristics such as its space and time distribution, evaporation, infiltration, and land use characteristics which, in turn, include man's influence including afforestation, deforestation, cropping pattern, cultivation practices etc. This classification is by no means exact because many of the factors are interdependent to a large extent and some of the factors may be permanent or transient. In order to estimate the flood magnitude, it is important to study the influence of various factors on the flood hydrograph and its peak.

2.3 Classification of Regionalisation Procedures

Regional flood estimation procedures are used for ungauged sites or when the available data at the site are inadequate. A simplistic approach is generally to develop graphs or equations which may be used to estimate the peak discharge or flood hydrograph for any catchment in the region. Such empirical relationships should be used with care because they often depend on particular physical and climatic conditions not accounted for in the relationships and so they are applicable at best only to those regions for which they are developed . Depending on the purpose and availability of data, the procedures for data analysis and flood estimation may vary.

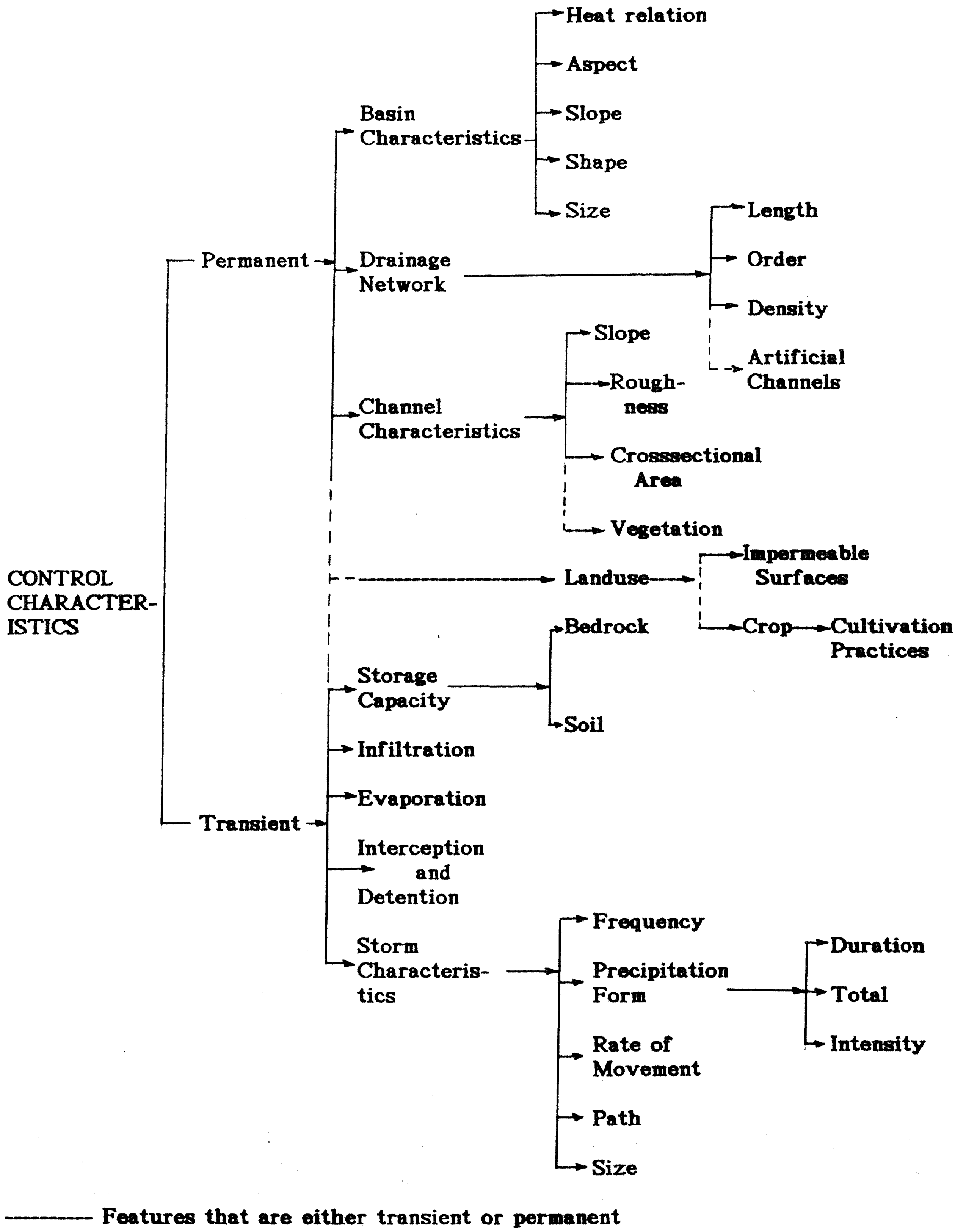


Fig.2.1 Controls of flood hydrograph characteristics (Rodda, 1969)

In many cases, an estimate of peak flow rate is sufficient for engineering design purposes, e.g., for design of bridges, culverts, etc. But for others, particularly those involving the provision of storage capacity, a complete flood hydrograph may be necessary. Hence, regionalisation procedures may broadly be classified into:

a) Flood peak estimation procedures, and

b) Flood hydrograph estimation procedures.

They are briefly considered in the following sections.

2.4 Flood Peak Estimation Procedures

Many hydraulic design problems require simply an estimation of the peak flow rate in a stream, perhaps, for a specified frequency. Then, the general shape of the hydrograph and the time of occurrence of the peak flow are of no special significance. Approaches for estimating peak flood discharges fall into one of the two broad categories, viz., (i) deterministic methods and (ii) probabilistic methods. Fig. 2.2 shows the various approaches for peak flow estimation and they are considered in greater detail here.

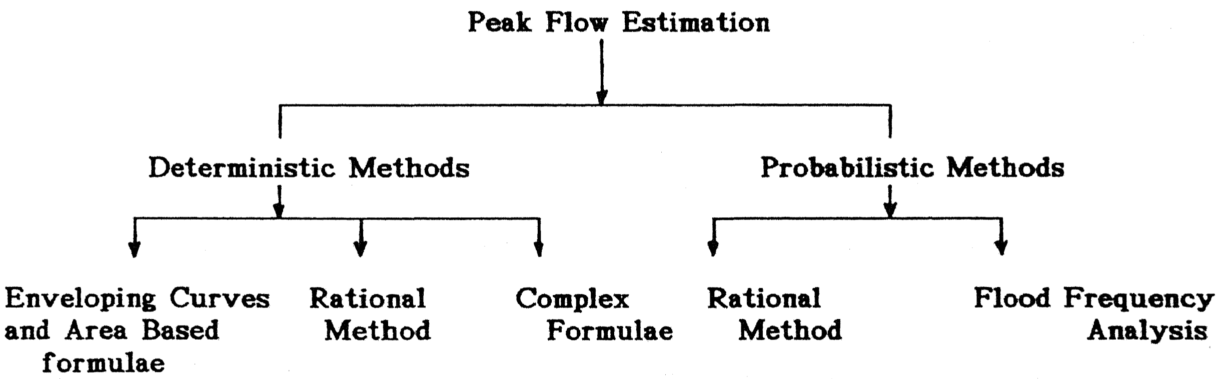


Fig.2.2 Approaches for flood peak estimation

2.4.1. Deterministic methods

(i) Enveloping curves and area based methods

The catchment area of a basin is hydrologically important because it directly affects the magnitude of flood peaks. The larger the size of the basin, the greater is the amount of rainfall intercepted and higher is the peak discharge that results. The observed maximum peak discharges for a number of catchments in a region are plotted on a log-log scale against the catchment area. The plotted points are then enveloped by a smooth curve referred to as the enveloping curve for observed maximum peak discharges. Two of the well known enveloping curves are those of Creager (Creager and Justin, 1950) and Meyer-Jarvis. Enveloping curves have been used as a guide for determining flood discharges in some projects in India like the Nagarjunasagar in Andhra Pradesh and Matatila in Uttar Pradesh (CWC, 1972). A typical enveloping curve as reported in CWC (1972) for Indian rivers is shown in Fig.2.3(a) and for different parts of the world (Gray,1970) in Figs. 2.3(b) and 2.3(c). Mutreja(1986) has given details of enveloping curves for Romania and U.S.A.

This method is based on available records and for maximum observed flood peaks only. The enveloping curves have to be reconstructed as more and more data become available since the maximum observed flood will be larger, the longer the period of record. It can hence be used only as a guide for the order of magnitude of the largest possible flood in a basin.

A functional form may be fitted to the enveloping or other curves relating peak discharge to the area. These equations are generally of the form

$$Q = C\,A^{n} \tag{2.1}$$

where Q is the peak discharge, A is the basin area and C and n are constants.

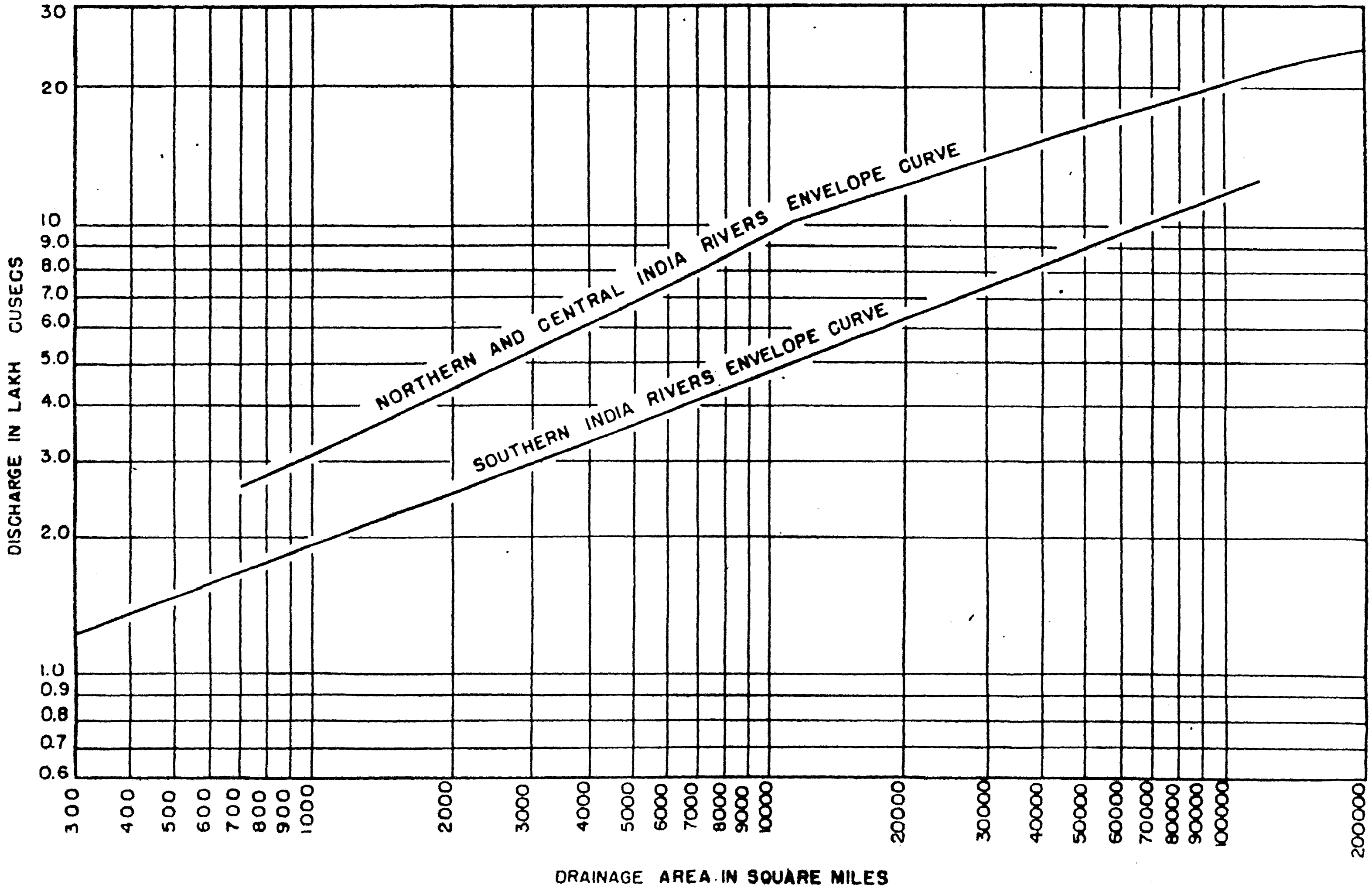


Fig. 2.3 (a) Enveloping curves for Indian rivers (CWC, 1972)

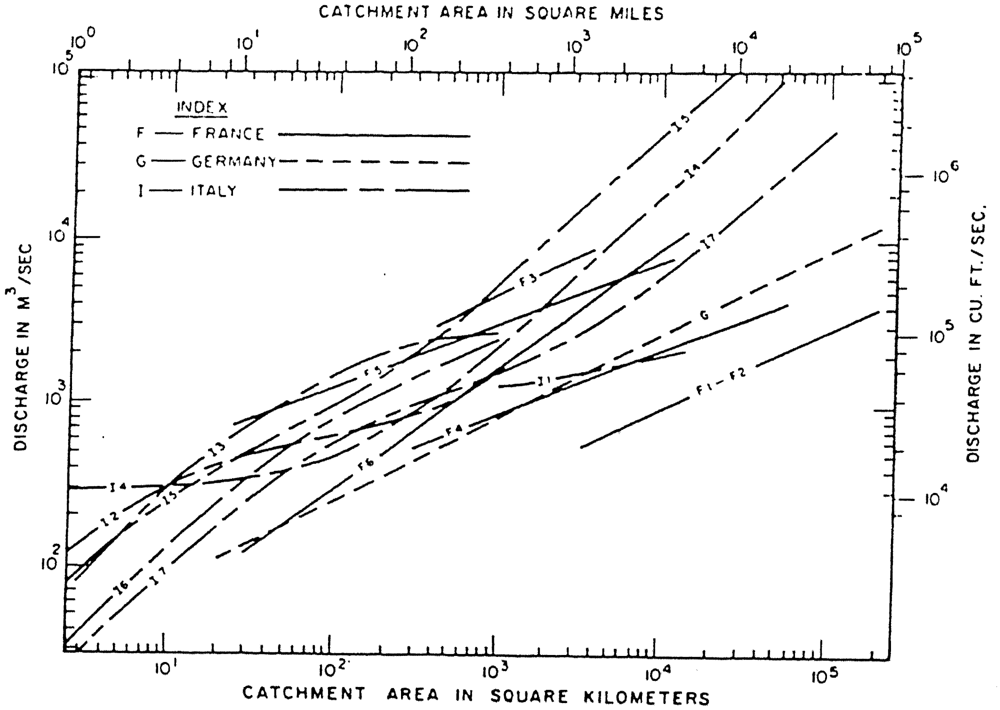


Fig.2.3 (b) Comparison of Flood Formulae (Gray,1970)

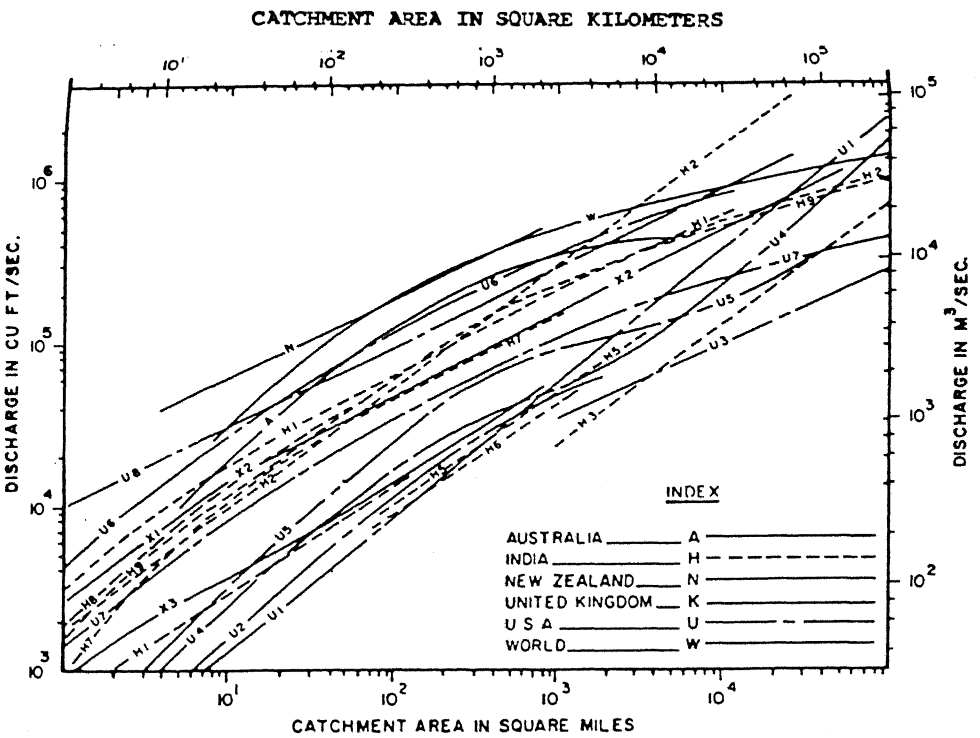


Fig.2.3. (c) Comparison of Flood Formulae (Gray,1970)

Dickens in 1885 (CWC,1972) made the first attempt in India to derive a general formula for determining maximum flood discharge on the basis of studies for determining the waterway capacities for bridges. His formula is

$$Q = C\,A^{3/4} \tag{2.2}$$

where Q is the discharge and A is the area. Mutreja(1986) presents a map of India which shows the values of Dicken's constant C to be adopted in different parts of India. Chow (1964) and Ghosh (1986) have listed various area based formulae for flood peak estimation and Wolf (1966) gives an excellent review of early relationships. The manual on "*Estimation of Design Flood - Recommended Procedures*" (CWC,1972) lists a wide variety of empirical formulae. Table 2.1 gives a number of area based formulae used in India and elsewhere in the world (Gray,1970).

The variation in the empirical formulae may be due to limited data, differences in curve fitting procedures adopted and because the data in each case are limited to a specific region. Hence such empirical relationships should be applied with care and to the specific region for which they may be relevant.

(ii) Rational method

The rational method assumes that the intensity of rainfall is an important parameter in the estimation of flood peaks and so estimates the peak runoff rates from data on rainfall intensity, catchment area and drainage characteristics. The method assumes that a rainfall of uniform intensity occurs for a sufficiently long time over the whole basin and losses and runoff are constant proportions of the rainfall. Furthermore, rainfall over different parts of the basin take different times to reach the gauging stations and so runoff will increase as the water from

Table 2.1 Comparison of formulae for estimating flood discharge (Gray,1970)

Sl. No.	Country	Equations	Units	Particulars about Eqns.	Curve Designation (Fig.2.3 b&c)	Author (or) Source
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.	The World	$Q_m = \frac{131{,}000A}{(107+A)^{0.78}}$	E	Max. recorded flood throughout the world	W	Baird and McIllwraith
2.	Australia	$Q_m = \frac{222{,}000A}{(185+A)^{0.9}}$	E		A	-do-
3.	France	$Q = (10 \text{ to } 70)A^{0.65}$	M	Mild rain, A, between 3,000 and 160,000 Km^2	F_1F_2	From a paper by A.Contagne
		$Q_m = 150\ A^{0.5}$	M	Violent rain A, between 400 and 3,000 Km^2	F_3	-do-
		$Q_m = 54.6\ A^{0.4}$	M	River Garonne	F_4	From a paper by A.Contagne
		$Q_m = 200\ A^{0.4}$	M	A, between 30 and 10,000 Km^2	F_6	-do-
		$Q_m = 10.76A^{0.737}$	M	Existing dams of "Massif Central"	F_6	-do-
5.	Germany	$Q_m = 24.12A^{0.516}$	M	A, between 15 and 200,000 Km^2	G	-do-
6.	India	$Q_m = \frac{7000A}{\sqrt{(A+4)}}$	E	For fan-shaped areas	H_1	Inglis
		$Q = 1{,}795A^{0.75}$	E	Rain approx. 100 inches	H_2	Dickens
		$Q = 149A^{0.75}$	E	Rainfall 30 to 40 inches	H_3	Dickens
		$Q = 675A^{0.67}$	E	Max. Flood	H_4	Ryves

Table 2.1 contd...

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	India (Contd..)	$Q = 560A^{0.67}$	E	Average Flood	H_5	Ryves
		$Q = 450A^{0.67}$	E	Min. Flood	H_6	Ryves
		Curve only	E	Bombay area	H_7	Whiting
		$Q = 2000A^{(0.92 - \frac{1}{15}\log A)}$	E	Tungabhadra River	H_8	Madras Formula (Paper by Rao, K.L.)
		$Q = 1750A^{(0.92 - \frac{1}{14}\log A)}$	E	Tungabhadra River	H_9	Hyderabad Formula (Paper by Rao, K.L.)
6.	Italy	$Q_m = (\frac{1538}{A+259} + 0.054)A$	M	A, between 1000 and 12,000 Km^2	I_1	Whistler (Paper by Tonni)
		$Q_m = (\frac{600}{A+10} + 1)A$	M	$A < 1000\ Km^2$	I_2	Seimemi (Paper by Tonni)
		$Q_m = (\frac{2900}{A+90})A$	M	-do-	I_3	Pagliaro (Paper by Tonni)
		$Q_m = (\frac{280}{A} + 2)A$	M	Mountain basins	I_4	Baratta (Paper by Tonni)
		$Q_m = (\frac{532.5}{A+16.2} + 5)A$	M	-do-	I_5	Giandotti (Paper by Tonni)
		$Q_m = (3.25\ \frac{500}{A+125} + 1)A$	M	$A < 1000\ Km^2$ Max. rainfall 400 mm in 24 hrs.	I_6	Forti (Paper by Tonni)
		$Q_m = (2.35\frac{500}{A+125} + 0.5)A$	M	Max. rainfall 200 mm in 24 hrs.	I_7	-do-
7.	New Zealand	$Q_m = 20{,}000A^{0.5}$	E	A < 10 sq.miles	N	Paper by Contagne

Table 2.1 contd...

(1)	(2)	(3)	(4)	(5)	(6)	(7)
8.	United Kingdom					
		$Q_m = 2,700A^{0.75}$	E	-do-	K_1	Bransby-Williams (Paper by Hunter and Wilmot)
		$Q_m = 4,600A^{0.52}$	E	A > 10 sq.miles	K_2	-do-
		Curve only	E		K_3	Inst.of Civil Engg. (Paper by Hunter and Wilmot)
9.	U.S.A.	$Q = 200\ A^{5/6}$	E		U_1	Paper by Rao, K.L.
		$Q = (\frac{46790}{A+320}+15)A$	E	$5.5 \leq A \leq 2000$ sq.miles	U_2	-do-
		$Q = 1,400A^{0.476}$	E	A, between 1000 and 24,000 sq.miles	U_3	U.S.G.S.for Columbia (Paper by Contagne)
		$Q = (\frac{44000}{A+170}+20)A$	E	For frequent floods	U_4	Kuichling (Paper by Rao, K.L.)
		$Q = (\frac{127000}{A+370}+7.4)A$	E	For rare floods	U_5	-do-
		$Q = 4600A^{0.048}A^{0.048}$	E	Upper limit	U_6	Creager (Paper by Hunter and Wilmot)
		$Q = 1380A^{0.894}A^{0.048}$	E	Lower limit	U_7	-do-
		$Q = 10000A^{0.5}$	E		U_8	Myer (Paper by Hunter and Wilmot)

(Compiled from Fourth Congress on Large Dams, 1951 Vol.2)

Q_m = max.flood; M = Metric system. (Q in m^3/s, A in Km^2)

A = drainage area; E = English system. (Q in cfs, A in sq.miles)

more and more distant parts of the catchment reaches the outlet. When the whole area is contributing and the rainfall is uniform, a steady state is reached and discharge becomes a constant and maximum. Since the rainfall intensity generally decreases with duration, the maximum peak runoff will occur when the duration of the storm is around the time when the whole of the basin area starts to contribute to runoff at the gauging station. This is, in turn, estimated as the time of concentration of the basin given by Kirpich equation (1940),

$$t_c = 0.00032\ L^{0.77} S^{-0.385} \tag{2.3}$$

where t_c = time of concentration in hours, L = maximum length of travel in metres, and S = slope = H/L where H is the difference in elevation excluding rapids and falls between the farthest point in the basin and the outlet in metres.

The rational formula is given by

$$Q = 0.278\ C\ I\ A \tag{2.4}$$

where Q is the peak discharge in m^3/sec, C is the coefficient of runoff depending on the characteristics of the drainage basin, I is the intensity of rainfall in mm/hour for a duration equal to the time of concentration and A is the drainage area in Km^2. The choice of the value of C is the most critical factor in the use of the rational method. This coefficient has to account for various climatic conditions and physiographic characteristics of the basin and in particular for the characteristics of the design storm. In reality, it is not a constant and it varies from storm to storm in the same basin. The judgement required in estimating the value of C is, therefore, considerable. However, used with discretion, this method can provide a reasonable estimate of peak discharge. The value of C for different surfaces have been listed in Dunne and Leopold (1978) and Chow et al (1989).

Chow (1964) suggests a modified form of rational formula for small basins

given by

$$Q = 1.008 \frac{R_e A Z}{t} \tag{2.5}$$

where Q is the peak runoff in cfs, Z is the fraction of unitgraph peak for a duration 't' to the equilibrium discharge for a continuous rainfall excess at a rate of 1"/hour over an area of A acres and R_e is the rainfall excess in inches for a given duration of t hours for the basin . Comparing this with the rational formula, it is observed that the coefficient C is equal to 1.008 Z.

(iii) Complex formulae

The climate of a region often controls the soil and vegetation in a basin. It is related to precipitation and temperature or solar radiation. The rational formula explicitly considers only the area of the basin and the intensity of rainfall and all other factors are implicitly lumped in the factor C. In addition to the area and the intensity of rainfall, there are many other factors that control the magnitude of flood peak (Sec. 2.2). A large number of basin and storm factors have been incorporated into a wide range of flood formulae by means of standard statistical techniques producing expressions of the form

$$Q = a\, X_1^{\,b}\, X_2^{\,c}\, X_3^{\,d} \tag{2.6}$$

where Q is the peak discharge; a,b,c,d etc., are the regression coefficients; and $X_1, X_2 \ldots$ are the factors controlling the flood peaks.

These procedures require data pertaining to various factors for several basins in the region. Chow(1964) and Wolf (1966) list formulae of this type.

2.4.2. Probabilistic approaches

The various methods considered earlier for estimating flood peaks are deterministic. But the engineer or hydrologist is more often interested in the number of times a flood of given magnitude is equalled or exceeded in a specified period, i.e., he is more interested in estimating the flood magnitude with a given recurrence interval particularly for the analysis and design of medium and minor structures. In such a case probabilistic approaches are necessary for estimating the magnitude of flood peaks.

(i) Rational Method

The rational method can be considered as a probabilistic method. It is assumed that the rainfall intensity has a duration equal to the time of concentration of the catchment. The probability distribution of annual flood peaks under constant infiltration differs from that for infiltration which really varies from storm to storm (Ramaseshan, 1964). Yet as an approximation it is assumed that the average recurrence interval (ARI) of the computed maximum rate of discharge for the catchment under constant infiltration condition is the same as the ARI for the corresponding storm. Thus if it is required to calculate the flood peak with a specified recurrence interval, a storm of the specified recurrence interval and duration t_c is used in the calculation. Thus, when used as a probabilistic model, the rational formula takes the form (Pilgrim, 1989)

$$Q(Y) = 0.278\ C(Y)\ I(t_c,Y)\ A \qquad (2.7)$$

where Q is the discharge and I is the intensity of rainfall for a duration t_c with an average recurrence interval of Y years. C, being a function of Y, is not a constant. This formula converts a rainfall of ARI of Y years derived from

intensity-duration-frequency data for the region under study into a peak discharge with the same ARI. Both the design rainfall and peak discharge are derived from frequency analysis and hence this formula has nothing to do with a particular storm. Pilgrim (1989) used this approach for Australia and has found that it fulfills the requirments for design methods for small and medium catchments.

(ii) **Regional flood frequency analysis**

Estimates of likelihood of flood of a given magnitude occurring within a stated interval of time are commonly required for planning and design of hydraulic structures. Where stream gauging records of sufficient length are available at or near the point of interest, a frequency analysis of the historical floods is useful in estimating the likelihood of future floods with a given probability of occurrence. The flood parameters for which estimates of frequency are required may be the peak stage, peak discharge or volume of runoff.

Approaches to regionalisation are generally based on the assumptions that the frequency distribution applicable to flood peaks is unique, say Extreme Value, Log Pearson Type III etc., and that the parameters of the distribution, say, mean and standard deviation are related to the area of the basin and parameters like C_v (Coefficient of variation) and C_s (Coefficient of skew) may vary in space. Furthermore the parameters may depend on the length of available data.

Estimates of flood frequency are often required at ungauged sites for which no data are available. Regionalisation is useful in making such estimates. The record at any gauged site may not be of sufficient length to provide reasonably accurate parameter estimates. In such cases, information from nearby sites in the region may be correlated with the record of the site with data of inadequate

length to increase its information and to provide a regionalised estimate of flood frequency.

The index flood method devised by the United States Geological Survey and reported by Dalrymple (1960) ia a method of regionalisation of flood estimation . It attempts to extrapolate statistical information of runoff events for flood frequency from gauged catchments to ungauged catchments in the vicinity having similar catchment and hydrologic characteristics.

To illustrate the empirical nature of such approaches, the various steps of the index flood method are briefly listed:

i) Select gauged catchments within the region having characteristics similar to the ungauged catchments.

ii) Determine the time base period to be used in the study;

iii) Establish flood frequency curves for each gauging site using Gumbel Extreme Value-Type I distribution;

iv) Estimate the mean annual flood $Q_{2.33}$ for each station;

v) Test homogeneity of data by a specific procedure;

vi) Establish relationship of mean annual flood and catchment characteristics, usually drainage area, at each station;

vii) Rank ratios of selected return period floods to the mean annual flood at each station; and

viii) Compute median flood ratio for each of the selected return periods; multiply by the estimated mean annual flood of the ungauged catchment; and plot against the recurrence interval on Gumbel's probability paper to get the flood frequency curve for an ungauged catchment.

The regional flood frequency analysis for British Isles reported in the Flood

ood Studies Report (NERC,1975) employed a modification of Dalrymple's approach mparable to that of Benson(1962) (Hall,1981).

The usefulness of frequency analysis is restricted because observed flow ta for small catchments are usually very limited (Raudkivi,1979). Apart from the gional flood frequency analysis a large number of studies using statistical proaches have been carried out. Condie et al (1987) used the analysis of variance chnique to compare the regional flood frequency methods in Southern Ontario. heir study allows an assessment of the influence of drainage area and return eriod and they found that the drainage basin size has a significant effect on the esult of the study. Stedinger and Tasker(1985) proposed a method of estimating e parameters of regional regression models that take into account their length of ecord available at gauged sites and the between-site correlation among concurrent ows. This method uses the generalised least square estimator of the regression odel parameters. Several investigators like Shane and Garver (1970); Vicens, odriguez and Shaake(1975); and Kuczera(1982) have addressed the issue of how est to combine at-site and regional regression estimates. Mimikou and Gordio(1989) sed the multiple regression techniques in the development of regional relationships etween the basin characteristics and the mean annual floods. They successfully sed the model in predicting mean annual floods needed in the hydrologic design for ngauged catchments for the study region in Greece. Many more studies have been arried out for the regional flood frequency analysis (Singh,1987).

2.5 Flood Hydrograph Estimation Procedures

The time distribution of runoff during a storm is often needed for many

planning purposes. A hydrograph depicts the time distribution of runoff. It is used for engineering planning such as reservoir design, for assessing the flood damage potential and the influence of flood detention structures in reducing the flood peaks. The whole hydrograph is necessary when hydrographs from dissimilar tributary areas are added and routed downstream to a channel reach of interest. The duration of flooding, a critical factor in many planning problems, can also be studied from a knowledge of the hydrograph.

2.5.1 Rainfall - runoff relationships

From the hydrologic point of view, runoff from a drainage basin may be considered as an output from the watershed due to effective rainfall as input. The runoff is influenced by both physiographic and climatic factors. It is hence often necessary in hydrologic analysis and design to develop relationships between rainfall and runoff possibly using some of the factors affecting runoff. These relationships differ with the type of precipitation, consideration of the volume or peak runoff, or the time distribution of runoff. The need for such cause-effect relationships has lead to the development of hydrologic models. Such models help to predict the hydrologic output, for example, the runoff hydrograph from a given rainfall.

From available data, regression relationships are derived to relate peak discharge to hydrometeorological features of the storm and physiographic features of the watershed. Even when adequate data are available for establishing such relationships, they are limited to regions for which such relationships have been derived. Since generally adequate data are available for estimating storms, it seems worthwhile to derive relationships between storm rainfall and streamflow,

without bothering about the processes governing the water movement. Such models where no consideration is given to the internal structure of the system and where the model parameters and variables are lumped in space are termed as lumped parameter models (Eagleson,1967; Holton,1970). The development of lumped models can be stated to have started with the advent of the rational formula. Many models of the rainfall-runoff process treat the precipitation input as uniform over the watershed and ignore the internal spatial variation of the watershed flow. During 1960's and 70's, a large number of conceptual lumped models have been developed (Fleming, 1975).

A unit hydrograph representation is a classical example of lumped rainfall runoff system and since it was first introduced by Sherman (1932), in many situations it continues to be the most practical tool available to hydrologists for making flood estimates.

The unit hydrograph of a catchment is defined as the direct surface runoff (DSRO) resulting from unit excess rainfall over the drainage area generated at a constant rate during a specified duration. In the estimation of flood discharges from rainfall intensity and duration, the first concern is to find the amount of rainfall excess and the second is to obtain the flood discharge from the rainfall excess. With sufficient rainfall-runoff data, UH can be derived by various methods (Clarke, 1945; O' Kelly, 1955; Nash,1957; Dooge,1959).

a) Synthetic unit hydrograph

To derive a UH for sites where data are not available, it is important to have procedures by which UH can be constructed. For ungauged basins, the UH is synthesised from the physical characteristics of the basin.

Estimates of UH for an ungauged basin can be made if the information derived from the analysis of gauging records on other streams is regionalised. UHs are computed for the gauged streams in an area and the lag, peak and duration of these hydrographs are related to the hydrometeorological parameters of the catchment such as drainage area, channel gradient and drainage density. These physiographic and geomorphologic characteristics of the catchment represent the various factors that affect the storage and transmission of the temporally and spatially distributed runoff generated by a rainstorm and thereby control the time distribution of runoff. A set of regression equations is derived between the characteristics of the UH and geomorphological variables. Knowing the values of the geomorphological features of the basin from physical measurements and meteorological characteristics of the storm, flood peaks from ungauged basins can be estimated using regression equations. Several methods (Snyder,1938; Commons,1942; William, 1945; Taylor and Schwarz,1952; SCS, 1957; Bender and Roberson,1961; Gray,1961) have been developed for synthesis of UH.

i) Soil Conservation Service triangular hydrographs. From a study of UH for a large number of small drainage basins, SCS (1972) has developed modification of the synthetic unit hydrograph technique. The simple method is an approximation of the UH by a triangular unit hydrograph as shown in Fig.2.4.

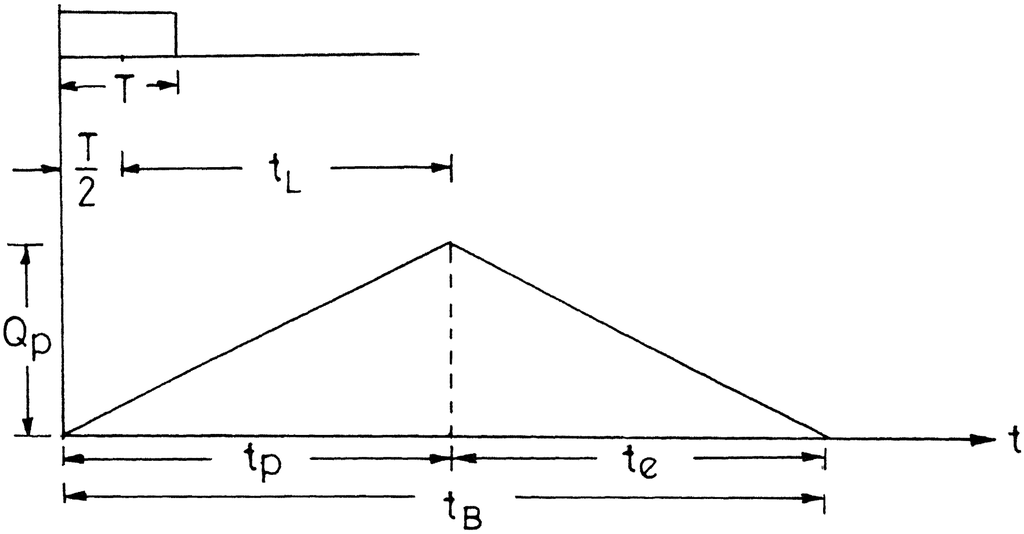


Fig. 2.4 SCS triangular hydrograph

From the area of the triangle, the volume of runoff V is equal to

$$V = \frac{1}{2} Q_p \, t_B \tag{2.8}$$

$$= \frac{1}{2} Q_p \, (t_p + t_e) \tag{2.9}$$

$$\text{or} \quad Q_p = \frac{2\,V}{t_p + t_e} \tag{2.10}$$

Examination of a large number of hydrographs from small agricultural watersheds throughout the United States of America lead to the empirical generalisation that

$$t_e = 1.67 \, t_p \tag{2.11}$$

which gives peak discharge, $Q_p = \frac{0.75\ V}{t_p}$, and time to peak $t_p = T/2 + t_L$.

ii) **SCS dimensionless hydrograph.** The SCS has also used a dimensionless hydrograph approach. The dimensionless hydrograph is a synthetic unit hydrograph in which the discharge is expressed by the ratio of discharge Q and peak discharge Q_p and the time by the ratio of time T to the time of rise of UH t_p. Given the peak discharge and the lag time for the duration of excess rainfall, the UH can be estimated from the synthetic dimensionless hydrograph of the basin.

The SCS triangular and dimensionless UH's employ a standard hydrograph form to all the catchments. If one does not want to rely on such a generalised procedure, he may adopt the SCS approach but derive dimensionless or triangular UH for his own region.

iii) Snyder's synthetic unit hydrograph. Snyder introduced the procedure for developing the UH for basin with no data. He correlated the timing and peak rates of hydrographs derived from basins in the Appalachian mountains with measures of their physiographic characteristics. He used three parameters to describe the hydrograph, viz., the time lag to peak, t_p; peak discharge, Q_p; and the baselength t_B. Snyder's equation in metric units for a rainfall excess of standard duration $t_r = t_p/5.5$ are as follows:

$$t_p = 0.75\ C_t\ (LL_c)^{0.3} \quad \text{hours} \tag{2.12}$$

$$Q_p = \frac{2.75\ C_p\ A}{t_p} \quad \text{m}^3/\text{sec, and} \tag{2.13}$$

$$t_B = 3 + 3\ (t_p/24) \quad \text{days} \tag{2.14}$$

where L is the length of the main stream in Km. from the outlet to the stream divide; L_c is the distance in Km. from the outlet to a point on the stream nearest to the centroid of the catchment; and C_t and C_p are constant coefficients derived from the gauged watersheds in the same region. For a different duration t, the lag time was adjusted by

$$t_{PR} = t_p + (t - t_r)/4 \tag{2.15}$$

Q_{PR} and t_{BR} are calculated using this value of t_{PR}. With these values, the shape of the hydrograph may be sketched. The values of Snyder's coefficients C_t and C_p have been found to vary considerably depending on the topography, geology and climate. Although Snyder indicated that the coefficient C_t is affected by basin

$$q_p = 2.272\ (L\ L_c/S)^{-0.409} \quad (2.17)$$

$$t_p = 2.167\ (q_p)^{-0.940} \quad (2.18)$$

$$W_{50} = 2.084\ (q_p)^{-1.065} \quad (2.19)$$

$$W_{R50} = 0.856\ (q_p)^{-0.865} \quad (2.20)$$

$$W_{R75} = 0.440\ (q_p)^{-0.918} \quad (2.21)$$

$$t_B = 5.428\ (t_p) \quad (2.22)$$

$$t_m = t_p + t_r/2 \quad (2.23)$$

$$Q_p = q_p \times A \quad (2.24)$$

where q_p = Peak discharge of UH/unit area in m^3/sec;

t_p = Time to peak of UH from centre of unit rainfall duration, in hrs;

W_{50} = Width of UH measured at 50% peak discharge ordinate, in hrs;

W_{75} = Width of UH measured at 75% peak discharge ordinate, in hrs;

W_{R50} = Width of rising side of UH measured at 50% peak discharge ordinate, in hrs;

W_{R75} = Width of rising side of UH measured at 75% peak discharge ordinate, in hrs;

t_B = Base width of UH , in hrs;

t_m = Time to start of rise to the peak of UH, in hrs;

Q_p = Peak discharge of UH in m^3/sec; and

S = Equivalent stream slope in m/km.

Similar relationships have also been derived for other subzones in the country.

b) Use of conceptual models

Various methods are available for deriving unit hydrographs (subec.2.5.2). In order that the UH can be regionalised, it is necessary to represent the UH in

terms of a limited number of parameters, perhaps in terms of conceptual model very useful and simple model for UH is the one proposed by Nash (1957). Stu have been carried out (Singh,1976; NIH,1985) to compare the various concep models and it has been found that the agreement between the observed predicted hydrographs was quite good for the Nash model *vis a vis* the ot models. Nash model has two parameters N and K. Nash (1960) applied his model British catchments. He showed that the two parameters of his model were rela to the first and second moments of the IUH about the origin as

$$m_1 = NK \text{ ; and} \qquad (2.$$

$$m_2 = 1/N \qquad (2.$$

These two moments were than correlated empirically with the watersh characteristics as

$$m_1 = 27.6\ A^{0.3}\ S_0^{\ 0.2}\text{; and} \qquad (2.2$$

$$m_2 = 0.41\ L^{-0.1} \qquad (2.2$$

where S_0 is the overland slope in parts per 10,000 calculated as mean of a gri sample of slopes, A is the watershed area in square miles and L is the length o the channel in miles.

Two approaches are possible for using the UH method. One can study the storm in a given catchment and the resulting flood hydrographs, isolate or derive the UH for the catchment and then use it for estimation of design floods on the same catchment. In the second approach, UH from various catchments are taken and an attempt is made to correlate the hydrograph parameters with the storm and catchment characteristics.

In the present study, though many methods are available for the UH approach, it was decided to use the Nash model because it is simple and a number

of computer programs have already been implemented at IIT Kanpur. This model also well established and has been found to be efficient. Hence this model is us for the derivation of unit hydrographs, for regionalisation of the model paramet and hence for flood estimation for small catchments in the region. A bri description of the Nash model is presented in Chapter 4.

2.5.2 Distributed parameter models

A hydrologic system can be called distributed if its parameters and variabl are distributed in space (Eagleson,1967; Nash,1969; and Holton,1970). Distribute parameter models are physically based and the hydrologic processes ar represented by a set of partial differential equations, interrelated by the concept of continuity of mass and momentum. Freeze and Harlan (1969) suggested blueprint for physically based models. This blueprint suggests the types of input and outputs required for such physically based hydrologic response models. The output from such a response model would provide a total picture of the hydrologic system. This output would include the streamflow hydrographs at any point within the basin, the groundwater flow pattern and the soil moisture regime. The output would also incorporate surface water, soil water and groundwater zones as components in a single system, not as discrete elements. The blueprint suggested by Freeze and Harlan forms the basis for many physically based distributed parameter models.

In the distributed parameter models, the catchment is divided into a number of segments and each segment is simulated separately and the results combined to obtain the catchment response. Increasing the number of segments allows a more detailed description of the basin, but at the same time increases the computer

processing time in direct proportion to the number of segments. Moreover with multiple segments, determination of some parameters may become somewhat more difficult. Computation with details of the distributed parameter model is usually time consuming. However such a model is useful for research on the consequence of heterogeneity in a catchment. It is appropriate to ask under what conditions and for what type of problems is it advantageous to use a distributed parameter model. The choice of a model for a particular problem is never a simple one. It is inevitably based on economic constraints and personal experience and constraints, as well as purely hydrological consideration and scientific vigour. Data availability is often a crucial factor in such decision making (Beven, 1985).

Beven and O'Connel (1982) have considered the role of distributed parameter models in hydrology in some depth. Four major areas which offer the greatest potential for the application of distributed models have been identified by them as:

i) forecasting the effect of land use changes;

ii) forecasting effects of spatially variable inputs and outputs;

iii) forecasting the movements of pollutants and sediments; and

iv) forecasting the hydrologic response of ungauged catchments where no data are available for calibration of lumped models.

Thus, in particular, the following problems may need distributed parameter models:

i) Hydrology of urban and small watersheds for runoff estimation;

ii) Groundwater modelling;

iii) Estimation of soil erosion, transport, deposition and sediment behaviour;

iv) Surface water - groundwater interaction;

v) Water quality simulation; and

vi) Human impact on environmental processes dealing with the above.

All of these problems allow distributed models to exploit their majo advantages over other models. The parameters of the model are physicall; based and they can generally be either measured in the field or derived froı field measurements. In addition, the parameters as well as input data an(derived variables are distributed over space and time. The physical basis oi the model parameters also give some hope that the measured parameter value: may be extrapolated to other locations or other time periods.

The development of distributed parameter modelling of catchment hydrology has been a slow faltering process (Beven, 1985). There have been many studies on modelling individual processes, especially groundwater flows, unsaturated soil water flow and channel routing; but models involving interactive processes and the application of catchment scale models to real world problems are limited. Examples are those of Betson (1979), Dagan (1979), Freeze (1971,1980), Jayawardhane and White (1977,1979), Lane and Woolhiser (1977), Morris and Woolhiser (1980), Narasimhan and Witherspoon (1977) etc. Fleming (1975) lists some of the distributed parameter models that were developed in the sixties and early seventies. Dawdy et al (1978) developed the USGS Distributed Routing Rainfall Runoff model (DR3M). The computer program for the model for routing urban flood discharges through the branched system of pipes or natural channels using rainfall as input has been well documented. The model combines the soil moisture accounting and rainfall excess components developed by Dawdy et al (1973) with the kinematic wave routing method presented by Leclerc and Schaake (1973). Other models that have been developed include System Hydrologique European (SHE) developed in Denmark, Sweden and Britain (Beven,1980), USDA Agricultural Research Service Small

Watershed Model and the Institute of Hydrology Distributed Model (IHDM Morris,1980). There are in addition a number of models available that treat onl interacting infiltration and surface flow process (e.g., Huggins and Monke,196 Smith and Woolhiser, 1971; Engman and Rogowski,1974; Ross et a 1979;Kutchment, 1980). Beven and Kirby (1979) developed a physically base model which had certain parameters that are measurable in the field.

Predicting the response of ungauged catchments is one area of application where a physically based model can play an important role, since parameter values may be measured or estimated on the basis of more readily available information. There have been relatively few studies where such models have been applied to catchments, treated as if they were ungauged, with the prediction later compared to available measurements. Beven (1985) reports on the Finite Element Storm Hydrograph Model (FESAM; Li et al, 1977; Ross et al, 1979; and Shanholz, 1981) which is based on a Hortonian infiltration excess concept of runoff generation.

2.6 Models Selected for the Study

The present study is essentially a regional case study for flood estimation in small catchments. The data available for the study (Chapter 3) are very limited and hence regional flood frequency studies are not possible. Though the regression and empirical models of CWC and RDSO are available, they are not used in the present study because these relationships have essentially been developed for large catchments which have adequate and appropriate date. Since generally adequate data are available for estimating design storms, it seems worthwhile to derive relationships between storm

rainfall and streamflow, perhaps in terms of UH and then regionalise the model parameters for flood estimation for small catchments in the region. A well tested and established UH model, the Nash conceptual model is used in the present study (Chapter 4).

A more realistic approach would be to model the catchment runoff process according to physical principles. Information regarding physiography, soils, vegetation and the drainage pattern of the catchment is used in the physically based models. Though many other models are also available (Yen, 1989) which are physically based (Subsec. 2.5.2), a well tested model, the USGS DR3M is used in the present work to study the regional variation, if any, of the parameters of the model and also to judge their validity for flood estimation (Chapter 5).

CHAPTER 3

MODEL SELECTION AND PRELIMINARY ANALYSIS OF DA

3.1 Introduction

Since there are a large number of models generally available for hydrologic studies (Chapter 2), it is necessary to choose one or more of them for detailed analysis. This involves both the choice between competing models and procedure for estimation of optimum value of the parameters of the models chosen. The choice of a particular model depends on many factors like the purpose, data availability and the ease with which one can use them.

3.1.1 Criteria for model selection

World Meteorological Organisation Commission on Hydrology (WMO,1975) has recommended the following criteria for selection of models in various hydrological forecasting situations:

i) General and specific purpose and benefits of forecast (eg., continuous hydrograph of flood discharge, water quality etc;)

ii) Climate and physiographic characteristics of the basin;

iii) Length and records of various types of data;

iv) Quality of data both in space and time;

v) The availability and size of the computer, both for development and operation of the model as well as possible use of model by relatively non-expert hydrological forecasting personnel.

vi) The ability of the model to be conveniently updated on the basis of current hydrometeorological conditions.

The models chosen for the present study are the Nash model and the USGS DR3M. These models were chosen with a specific purpose. They are both easy to use and the computer programs for these models were available in the computer system used for the study. The parameters of these models are used for regionalisation for flood estimation. Since the data available are for a limited period, the regional flood frequency analysis or any other statistical approaches cannot be used. With the available data, the two models chosen can be applied to the various basins in the study region. Though many computer systems were available for the study, the entire computational work was carried out on the microVAX II system of the Centre for Water Resources Engineering and Management, IIT Kanpur. The model parameters can be updated with the availability of additional data.

3.1.2 Steps in model selection

Dooge (1973) gave a rational methodology for the selection, calibration and use of mathematical model for the runoff process that might follow a procedure as shown in Fig. 3.1. The first step is to define a problem (Chapter 1). The next step involves choosing class of models and selecting a model. The present study uses a lumped parameter model and a physically based distributed parameter model . The models chosen are the Nash model and the USGS DR3M. The process is explained in

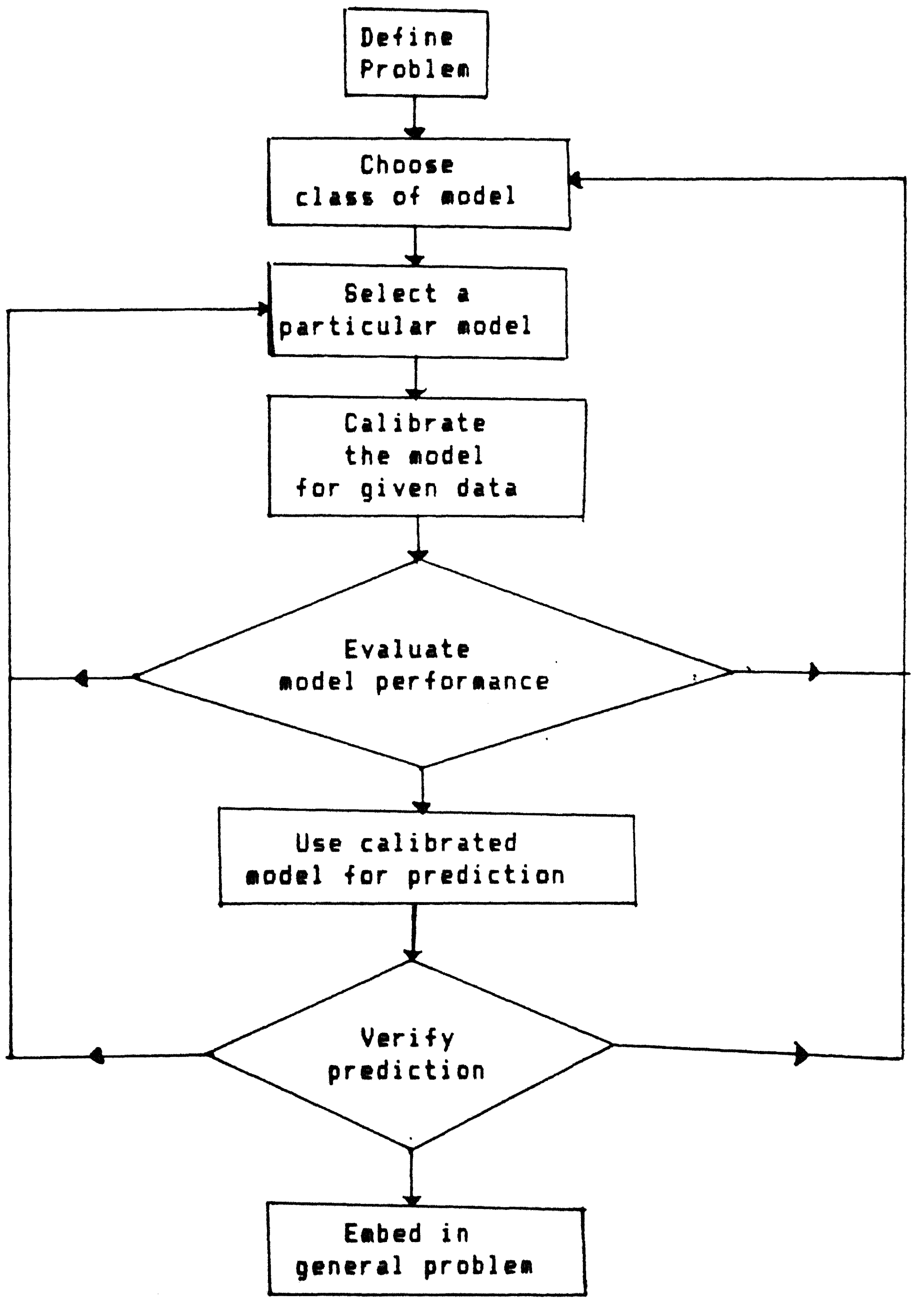


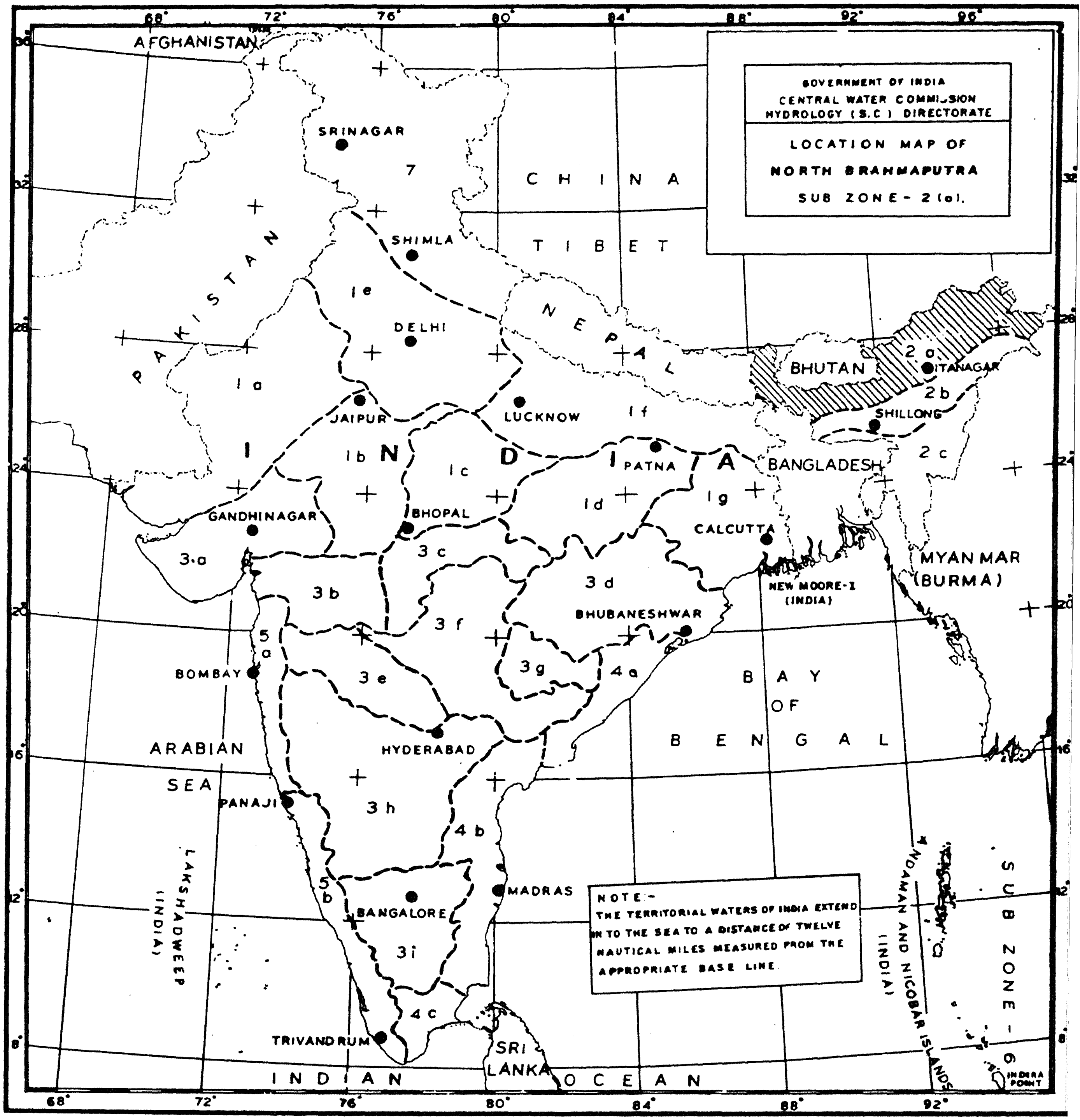
Fig. 3.1 Methodology for mathematical modelling

Chapter 2 and 3. The models are used for the various basins with the available set of data and their performance is evaluated and used in flood estimation (Chapters 4 and 5). The results are summarised in Chapter 6 along with the discussions on the suitability of the selected models for the given problem. Thus, in general, the procedure followed in this study follows the method suggested by Dooge.

3.2 Study Area

The various basins used in a regional study have to be generally similar in hydrometeorological characteristics. However, it is usually very difficult to specify the criteria for regional homogeneity even though some empirical tests have been suggested (Dalrymple,1960). The region selected for the study may be based on empirical criteria. For purposes of hydrometeorological studies, India has been divided by the Central Water Commission (CWC) into 26 subzones, each of which is assumed to be homogeneous and these are shown in Fig. 3.2. The present work is essentially a regional case study for flood estimation, especially in small catchments. RDSO of the Ministry of Railways has collected and in collaboration with CWC has analysed a large amount of hydrometeorological data for floods for a large number of catchments draining into various railway culverts and bridges in different subzones. After discussion with the officers of the RDSO, a part of the North Brahmaputra basin, designated as subzone 2(a) was identified for this study. It may be noted that the data available for small catchments in this region are yet to be analysed and interpreted.

The subzone lies approximately between longitudes 88^{0}E and 97^{0} 20' E and latitudes 26^{0}N and $29^{0}25'$N. Actually the various small basins for which data were



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RESPONSIBILITY FOR THE CORRECTNESS OF INTERNAL DETAILS RESTS WITH THE PUBLISHER.

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S.N. MALHOTRA , L.P. NAUTIYAL.

FIG. 3.2 LOCATION MAP OF NORTH BRAHMAPUTRA SUBZONE

available and are used in the present study lie in a part of this subzone between longitudes 89^0E and 93^0E and latitudes 26^0N and 27^0N (Fig. 3.3).

The Brahmaputra river with a total catchment area of 0.94 million Km^2 drains one of the largest river basins in the world. The total length of the river in India is 885 km. The drainage area of the river in India is 1,95,000 Km^2. The drainage area of the North Brahmaputra subzone 2(a) is 1,21,444 Km^2 and the South Brahmaputra subzone 2(b) is 73,556 Km^2. The topography of the area is generally hilly with rivers having moderate steep slope, meandering channels, beds and bank of alluvial soils and comparatively low silt charge. The south-west monsoon and cyclonic storms cause rainfall in the subzone from May to October. Normal annual rainfall in the basin varies from 2000 mm to 5000 mm. The subzone is covered with a variety of soils. Broadly they can be classified as red loamy soil, brown hill soil, Terai soil and alluvial soil of recent origin. The subzone has a considerable area under forests which may have undergone changes in the recent times because of human activities.

3.3 Data Availability

3.3.1 Available data

The data available for various river basins in the subzone include:

i) Catchment plan;

ii) Crosssection at bridge site;

iii) Longitudinal section maps for some of the catchment selected;

iv) Hourly rainfall data from raingauges at one or more self recording raingauge stations for the period, generally from middle of May to middle

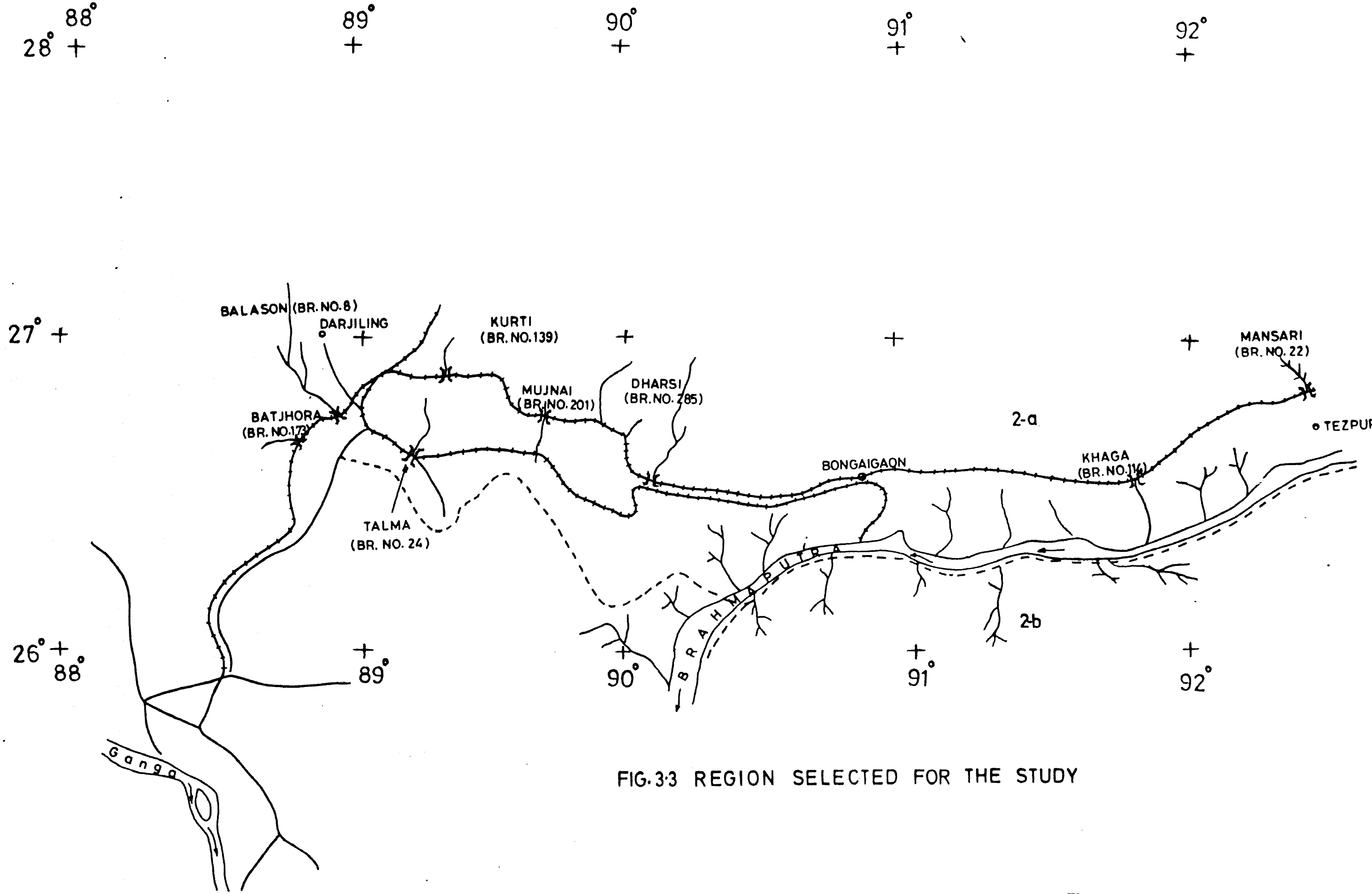

FIG. 3·3 REGION SELECTED FOR THE STUDY

of October;

v) Hourly gauge data at bridge site; and

vi) Data for rating curves in terms of discharge at gauging sites through velocity observations, using generally floats, two to three times a day;

3.3.2 Source of data

North-East Frontier Railway under the supervision and guidance of RDSO had observed and collected hydrometeorological data for the subzone 2(a). Flood estimation procedures for the larger catchments have been developed using the unit hydrograph principle (CWC,1991). Since data and procedures for the large basins are generally available, RDSO is interested in the development of a procedure or procedures for estimation of design flood for railway bridges and culverts, particularly for small catchments. Some information was also obtained from the Small Catchment Directorate of CWC. Meteorological data were gathered from the IMD and other publications, e.g., Irrigation Atlas of India; Report of the National Commission of Floods (RBA, 1980) etc.

3.3.3 River basins selected for the study

The entire subzone, being in a very difficult terrain, had disadvantages of inadequate accessibility and communication. In general the data for the small catchments in the region pertain to the period 1961 to 1966. During this period, there were two major wars and there were no observations. Hence, this area suffered from disturbed conditions during the period of observation. Due to these reasons, collection of data and supervision of the same could not be of adequate

standard.

Based on the available information and after initial scrutiny of data, the following river basins were identified for use in the present study:

Table 3.1 River basins identified for the study

Sl.No	River Basin	Bridge No.	Catchment Area (Km^2)	No.of Rain gauges	Theissen Weights
1.	Batjhora	173	7.33	1	1.00
2.	Sankhini	112	7.76	1	1.00
3.	Kurti	139	21.91	1	1.00
4.	Mujnai	201	38.50	2	0.24;0.76
5.	Talma	24	42.12	1	1.00
6.	Khaga	114	66.00	2	0.46;0.54
7.	Dharsi	285	91.40	2	0.23;0.77
8.	Mansari	22	213.00	2	0.24;0.76
9.	Gaghra	506	13.02	1	1.00
10.	Balason	8	350.20	4	0.11;0.27;0.38;0.24

3.4 Preliminary Analysis

For both the models used in the study, rainfall that causes the flood and the corresponding streamflow values are required in addition to the catchment details. A close scrutiny of the available data indicated inconsistencies and errors and these have to be taken care of before using the data in the two models. The following sections briefly explain the various preliminary analyses carried out and the decisions taken.

3.4.1 Precipitation data

Precipitation data were available at one or more locations in each basin. The

data were, in general, measured using self recording raingauges and tabulated hourly rainfall values were made available for the study. For 4 basins, there was only one raingauge station. In other cases, where there were more than one raingauge in a basin, the Theissen polygon method was used to estimate the average rainfall over the basin. The average precipitation over the catchment is calculated as:

$$P_{av} = \frac{A_1}{A} P_1 + \frac{A_2}{A} P_1 + \ldots\ldots\ldots\ldots\ldots + \frac{A_n}{A} P_n \tag{3.1}$$

where P_{av} = Average precipitation over the catchment;

$A_1, A_2, \ldots\ldots A_n$ = Area within each Theissen polygon with corresponding rainfalls P_1, $P_2, \ldots\ldots\ldots P_n$; and

A = Total area of the catchment.

The Theissen weights for all the basins are given in Table 3.1.

3.4.2 Stage data

A detailed study of the available data indicated large errors and inconsistencies. The errors were mainly in the stage-discharge observations. The flood events were identified based on the rise and fall in the stage of the river at the gauging site. On many days, night observations were not available and even if they were available, they showed significant inconsistencies. For the computation of discharges, velocities were observed using floats and hence were widely varying. In a single day, there were widely differing values of velocities for the same value of stage and vice versa. Further in a number of cases, on days with heavy rainfall, the observation sites, as per remarks in the record books, were inaccessible and hence stage data were not available. There were also remarks in the observation book, such as the instruments being tramped by animals like rhinoceros and

elephants. Thus very valuable data were lost for many days till the instruments got replaced.

After detailed scrutiny of records, including comparison of rainfall and runoff data, fairly consistent flood events were identified for the study.

3.4.4 Rating curve

It is difficult to measure continuously streamflows in rivers, but relatively easier to measure the stages or water levels continuously. Thus the establishment of a relationship between the rate of flow and water levels (and slopes if necessary) in channel, river etc., helps in conversion of stage data to discharge data. Water level in the river and corresponding discharge may be hydraulically related for uniform flow through Manning's equation or empirically related by plotting the measured discharge against the corresponding stage and establishing a regression relationship between stage and discharge. This relationship between stage and discharge is referred to as a rating curve and the relationship may be of the form

$$Q = K\,(H - H_0)^n \tag{3.2}$$

where K and n are the constants, and H is the stage while H_0 is the hypothetical datum corresponding to zero discharge.

While establishing the rating curve, it is to be noted that the discharge is generally expressed as a function of stage alone and corrections for slope are made where necessary. Logarithmic transformation of Eqn.3.2 leads to a linear relationship

$$\log Q = \log K + n \log (H - H_0) \tag{3.3}$$

In the present study, the electronic spreadsheet software Lotus 1-2-3, available for developing the rating curve (Tirupati,1989), is used. A worksheet is prepared using

stage and discharge data. For an assumed value of H_0, the built-in regression option is used for the estimation of regression parameters. The regression output in the worksheet gives the additive and multiplicative constants, the standard error of estimate etc. A sensitivity analysis is carried out to determine by the method of least squares, estimate of H_0 and the corresponding values of K and n., i.e., when the standard error of the $Q_{estimate}$ is minimum. The corresponding level H_0 is chosen as the datum level. A 95% confidence band is also computed. The logarithmic values may be transformed back to natural discharge values. Using the graph utility of **Lotus 1-2-3** , rating curve for the basin with 95% confidence band are plotted in logarithmic and discharge domain.

The rating curves are prepared for all the basins selected for the study. A typical worksheet is shown in Table 3.2. Titles and column headings are given in the top of the worksheet. The first two columns in the worksheet have the gauge and discharge data. The third column contains the difference between the gauge and datum level. The datum level selected is given in column 13. Columns 4 and 5 give the logarithm of the (stage - datum) and discharge values. A regression is carried out between the values of columns 4 and 5 with stage as the independent variable and the discharge as the dependent variable. The regression outputs are presented starting from column 12. Column 6 gives the values of the discharge calculated using the results of the regression. Column 7 and 8 have the 95% confidence band values calculated as equal to $Q_{estimate} \pm 1.96$ * standard error

TABLE 3.2 RATING CURVE FOR MUJNAI BASIN FOR THE YEAR 1964

CHARGE SECS	STAGE- DATUM	LN(H-HO) X	LN(Q) Y	Y compute	95% confidence level for Y		Q compute	95%confidence level for Q	
2	3	4	5	6	7	8	9	10	11
476	1.2	0.182321	6.165417	6.201486	6.355659	6.047313	493.4821	575.7421	422.9751
475	1.2	0.182321	6.163314	6.201486	6.355659	6.047313	493.4821	575.7421	422.9751
454	1.2	0.182321	6.118097	6.201486	6.355659	6.047313	493.4821	575.7421	422.9751
521	1.2	0.182321	6.255750	6.201486	6.355659	6.047313	493.4821	575.7421	422.9751
494	1.2	0.182321	6.202535	6.201486	6.355659	6.047313	493.4821	575.7421	422.9751
540	1.3	0.262364	6.291569	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
510	1.3	0.262364	6.234410	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
460	1.3	0.262364	6.131226	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
448	1.3	0.262364	6.104793	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
482	1.3	0.262364	6.177944	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
496	1.3	0.262364	6.206575	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
518	1.3	0.262364	6.249975	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
503	1.3	0.262364	6.220590	6.300363	6.454537	6.146190	544.7700	635.5793	466.9353
613	1.4	0.336472	6.418364	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
588	1.4	0.336472	6.376726	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
543	1.4	0.336472	6.297109	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
578	1.4	0.336472	6.359573	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
586	1.4	0.336472	6.373319	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
607	1.4	0.336472	6.408528	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
548	1.4	0.336472	6.306275	6.391909	6.546082	6.237736	596.9956	696.5105	511.6990
652	1.5	0.405465	6.480044	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
660	1.5	0.405465	6.492239	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
656	1.5	0.405465	6.486160	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
684	1.5	0.405465	6.527957	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
633	1.5	0.405465	6.450470	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
709	1.5	0.405465	6.563855	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
696	1.5	0.405465	6.545349	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
677	1.5	0.405465	6.517671	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
619	1.5	0.405465	6.428105	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
636	1.5	0.405465	6.455198	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
681	1.5	0.405465	6.523562	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
666	1.5	0.405465	6.501289	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
707	1.5	0.405465	6.561030	6.477137	6.631310	6.322963	650.1070	758.4753	557.2221
692	1.6	0.470003	6.539585	6.556861	6.711035	6.402688	704.0587	821.4203	603.4654
762	1.6	0.470003	6.635946	6.556861	6.711035	6.402688	704.0587	821.4203	603.4654

12 13

Datum level for which minimum Error in Y Estimate occurs= 296.5

Regression Output:

Constant		5.976263
Std Err of Y Est		0.078659
R Squared		0.969516
No. of Observations		113
Degrees of Freedom		111
X Coefficient(s)	1.235305	
Std Err of Coef.	0.020790	

HO	Std Err of Y estimate
296.00	0.082661
296.50	0.078659
297.00	0.084321

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of estimate. Columns 9,10 and 11 have the retransformed values of the discharge and the 95% confidence level values. A few iterations are carried out by changing the datum level and the value of H_0 corresponding to the lowest value of the standard error of $Q_{estimate}$ is chosen for preparing the rating curve. A plot of (stage - datum) against $Q_{estimate}$ and the 95% confidence band gives the rating curve for the basin with 95% confidence. A typical rating curve for Mujnai basin is shown in Fig. 3.4. (a). It is observed that a single rating curve does not fit the entire range of discharge and hence for this basin, two separate rating curves, one for the low flows and the other for the high flows are developed as shown in Fig. 3.4(b).

Furthermore, for some basins a single rating curve was sufficient for the entire period for which data were available. For others, for eg., Mujnai, different rating curves had to be used for different years because of the variation in the stage-discharge relationships over the years.

3.4.5 Evaporation data

No evaporation data were available. Hence using Christiansen's equation, pan evaporation values were calculated for Guwahati using monthly normal values of the meteorological characteristics obtained from IMD. Using a pan coefficient of 0.7 and assuming Guwahati data to be representative for the entire region, the daily potential evaporation values for the basins were estimated for use in the various analyses in the study.

3.5 Number of Floods in Each Basin

After detailed scrutiny of records, including comparison of rainfall and

Fig.3.4a. RATING CURVE FOR MUJNAI RIVER

1964 DATA NATURAL LOG VALUES

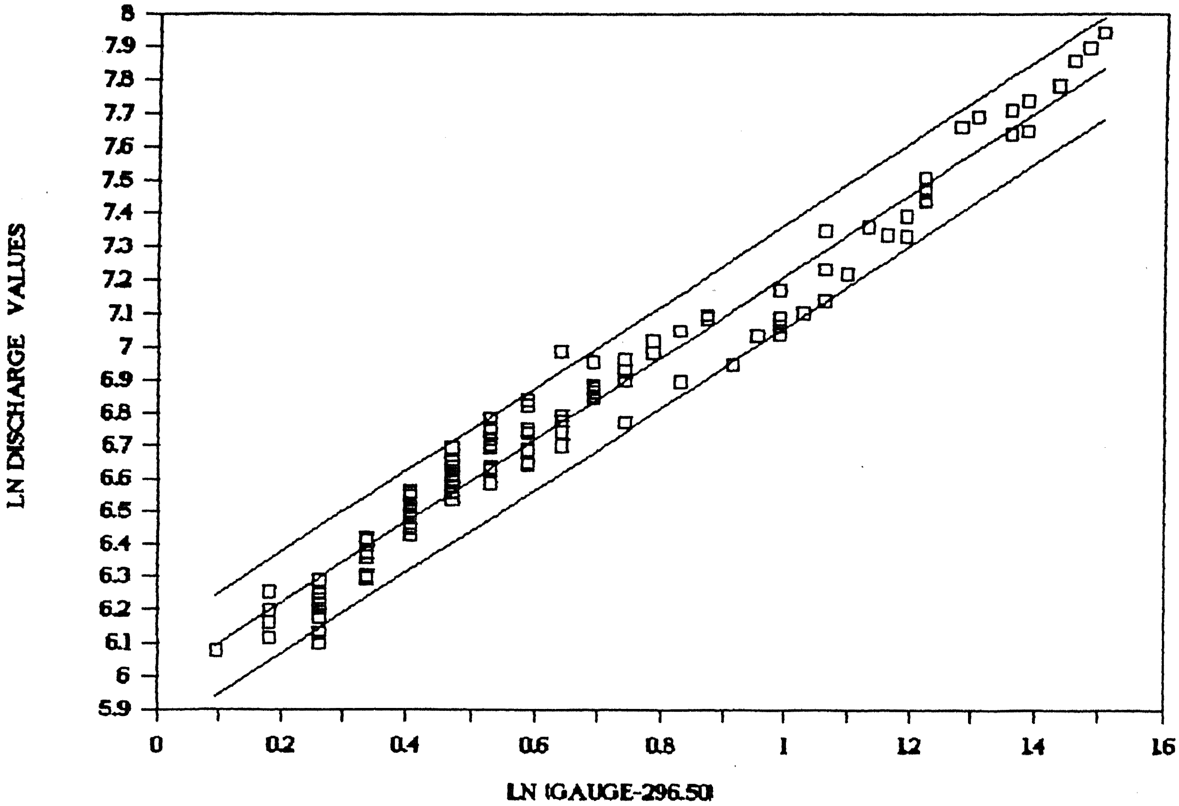
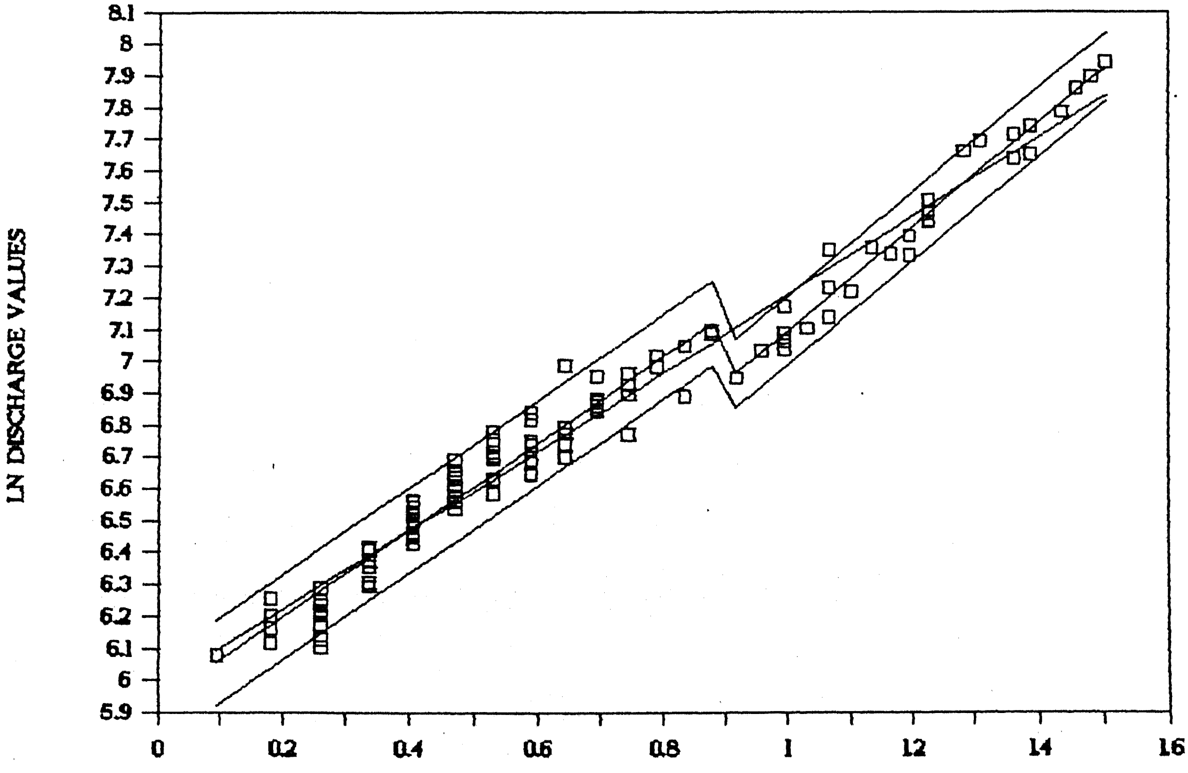


Fig.3.4b TWIN RATING CURVE–MUJNAI RIVER

1964 DATA NATURAL LOG VALUES



runoff records, consistent flood events were identified for the study. The number of floods in each basin is given in Table 3.3.

Table 3.3 Number of Floods in Each Basin

Sl.No.	Name of Basin	No. of floods
1.	Batjhora	11
2.	Sankhini	5
3.	Kurti	5
4.	Mujnai	13
5.	Talma	10
6.	Khaga	5
7.	Dharsi	18
8.	Mansari	5
9.	Gaghra	No suitable flood
10.	Balason	5

3.6 Conclusion

After the preliminary analysis, it was observed that Gaghra basin did not have any floods suitable for analysis because rainfall and runoff data were very erratic. The data for river Balason was highly inconsistent. In this case, on certain days of heavy rainfall, the stage data did not indicate any floods, while on days when the stage was on the rise, there were hardly any noticeable rainfall. Further the stage and discharge data also showed wide discrepancies and hence this basin was not taken up for the study. Thus after the preliminary analysis, the two basins Gaghra and Balason were rejected for the purpose of this study. Further analysis was carried out only for the remaining eight basins.

CHAPTER 4

REGIONALISATION USING UNIT HYDROGRAPH APPROACH

4.1 Introduction

Despite a large number of streams that are gauged, planning problems quite often involve ungauged basins. The estimation of peak runoff from a catchment is needed for comprehensive design of water control structures. The climatic and physical characteristics of the catchment are the major factors that influence the runoff. The climatic factors include precipitation, its duration, areal and temporal distribution, and intensity, evapotranspiration, interception etc. The physical characteristics which influence runoff are the area of catchment, its length, slope, shape, elevation, soil type, drainage system, vegetal cover etc.

Unit hydrograph (UH) is one of the most popular techniques for the computation of runoff from a catchment. A unit hydrograph may by defined as the hydrograph resulting from unit depth of surface runoff produced by a storm of unit effective rainfall and specified duration. It is considered to represent the integrated effect of the various physical factors on the routing of the effective rainfall through the catchment system. The UH approach assumes that the basin response is linear and hence hydrographs produced by different storms of uniform intensity and fixed duration but varying volumes of effective rainfall are essentially proportional to the volume.

For gauged catchments, the UH can be derived by analysing the available rainfall-runoff records. But when the stream is ungauged, the UH for the catchment has to be derived on the basis of regional relationships using their physical and

storm characteristics. This, in turn, necessitates the development of suitable regional relationships for estimation of unit hydrographs and hence estimation of flood. This is generally done by derivation of UH for gauged catchments for a given duration of effective rainfall and then by development of regional relationships between UH parameters with the physiographic and storm characteristics of the catchments. Thus, for development of regional relationships based on UH approach, the various steps involved would be:

i) Estimate the various UH parameters for a storm in a basin;

ii) Repeat the step for a number of storms in the basin to explain the variation, if any, of the parameters in terms of hydrometeorological characteristics; and

iii) Repeat the above steps for a number of basins in the region so that the parameters can be correlated to the physiographic characteristics of the basin.

4.2 Unit Hydrograph Analysis

The UH approach for determining the hydrograph of design flood assumes the catchment as a linear system which transforms the rainfall input into direct surface runoff (DSRO) as output. Application of this method involves the prior selection of storm and the estimation of loss or rainfall excess and the baseflow contribution.

4.2.1 Direct surface runoff separation

The total flow due to storm, in practice, can be considered to be consisting of

two parts, viz., storm or direct surface runoff (DSRO) and the baseflow. DSRO is the runoff resulting at the catchment outlet due to the rainfall excess and the baseflow is that contribution to the streamflow which results from release of water from surbsurface storage.

To get DSRO, the baseflow has to be separated from the total runoff. Many methods have been suggested (Chow,1964, Gray ,1970). Using a suitable method, the baseflow is separated and the DSRO hydrograph is obtained. In the present study, baseflow is considered as constant for a particular storm.

4.2.2 Effective rainfall

The part of the rainfall which appears over the surface as runoff and hence represents the rainfall contribution to the streamflow is called the excess rainfall. The rainfall excess can be determined by either of the following methods;

(i) The average rainfall over the basin for each time interval can be determined by arithmetic average, Theissen polygon method, isohyetal method etc. A constant abstraction rate is assumed at all time intervals. The excess of rainfall over abstraction rate is the effective rainfall and is otherwise zero. From principle of conservation of mass, cumulative effective rainfall during the storm should be equal to the total DSRO. The abstraction rate for the storm can be calculated by a trial and error procedure and this procedure is referred to as the infiltration index method.

(ii) If instead of taking the average rainfall over the basin, the effective rainfall for an abstraction rate is calculated for each of the station and the basin average of effective station rainfalls is calculated by any of the methods mentioned above, the effective rainfall will be more realistic. It is adopted

sometimes by the US Army Corps of Engineers and is referred to as the Corps of Engineers procedure.

The latter method is used in the present study if the number of raingauge stations in the basin is more than one. But when there is only one raingauge station in the basin, the two methods are identical.

4.2.3 Use of Nash model

Various models and methods are available for deriving the unit hydrograph for a basin. In order that the UH can be regionalised, it is necessary to represent the UH in terms of a limited number of parameters, perhaps in terms of conceptual models. A very useful and suitable model for UH is the Nash model. In the model proposed by Nash(1957), the watershed is represented by a series of N identical linear reservoirs, each having the same storage coefficient K. By routing unit impulse effective rainfall excess through N linear reservoirs, a mathematical equation for instantaneous unit hydrograph (IUH) for the model can be derived. The instantaneous unit hydrograph u(t) for a basin is its UH for an instantaneous unit effective rainfall. Nash derived the following equation for the IUH, which is mathematically a gamma function

$$U(t) = \frac{1}{K\,\Gamma(N)}\, e^{(-t/K)}\, (t/K)^{N-1} \tag{4.1}$$

where $\Gamma(N) = (N-1)!$ and is otherwise a constant which makes the integral $\int u(t)dt = 1$. The two parameters N and K determine the shape of the unit hydrograph. Nash generalised his model for noninteger values of N. $\Gamma(N)$ can be interpolated from tables of complete gamma function when N is not an integer (Abramovitz and

Stegun, 1965). A noninteger value of N is considered valid in this study.

4.2.4 Estimation of UH parameters for the Nash model

The parameters of the Nash model N and K are to be derived from data for defining the UH. Two methods, viz., (i) method of moments and (ii) method of least squares are generally used for deriving these parameters.

(a) Method of Moments: For an initially relaxed (i.e., Q(0) = 0.0) linear system, the output is related to the input through the UH by the convolution equation

$$Q(t) = \int_0^t I(t)\ U(t-\tau)\ d\tau \qquad 0 \leq t < \infty \tag{4.2}$$

where Q(t) represents the output direct runoff hydrograph (DRH); I(t), the input effective rainfall hyetograph (ERH) and U(t) is the IUH. Let X(t) be defined over $0 \leq t < \infty$. The k^{th} moment of a function X(t) about the origin is given by

$$M_k = \int_0^\infty X(t)\ \ t^k\, dt \tag{4.3}$$

Since different series may be given in terms of different measurement units, to avoid the need for standardisation of inputs and outputs in the same units the standard form of the moment equation is given by

$$M_k(x) = \frac{\int_0^t x(t)\ t^k\, dt}{\int_0^\infty x(t)\, dt} = \frac{\sum_{n=0}^{\infty} x(t)\ \Delta t\ t^k}{\sum_{n=0}^{\infty} x(t)\ \Delta t} \text{ for sampled data} \tag{4.4}$$

$$M_k(x) = \frac{\sum_{n=0}^{\infty} x(t)\ (t - \frac{\Delta}{2})^k}{\sum_{n=0}^{\infty} x(t)} \quad \text{for quantised data} \tag{4.5}$$

Rainfall is generally given as quantised data and the DSRO as sampled data. By taking moments of the convolution equation (4.2) and equating them, it is possible to derive various equations relating the moments of Q(t) in terms of those of I(t) and U(t). The first and second moments of U(t) for Nash model can be obtained from integration as $M_1(u) = N\,K$ and $M_2(u) = N\,(N+1)\,K^2$. Method of moments provides a simple approach to the calculation of the parameters of the IUH since the moments themselves can be expressed in terms of the parameters. For example, for Nash model

$$M_1(Q) = M_1(I) + M_1(u) = M_1(I) + NK \tag{4.6}$$

$$M_2(Q) = M_2(I) + M_2(u) + 2M_1(I)\,M_1(u) \tag{4.7}$$

with $M_2(u) = N\ (N+1)\,K^2$. For the two parameters of Nash model, only two moment equations are needed to uniquely define the parameters and can be solved to yield the values of N and K. This method is known as the method of moments approach to the estimation of the model parameters. Further details of estimation of these parameters for the sampled data Q and quantised data I are available in reference (Chow et al ,1988).

(b) Method of Least Squares: In this procedure, the parameters N and K are estimated by minimising the sum of the squares of the differences between observed and computed DSRO hydrographs. This procedure requires an initial estimate of the parameters and it estimates the parameters by searching in the parameter space for the minimum value of the objective function.

The method proposed by Nash is easily programmable. However, in the

present study, programs developed at the National Institute of Hydrology, Roorkee (NIH,1985) have been used with certain modifications. Two computer programs for estimating the parameters of the Nash model are available one each for (i) method of moments and (ii) method of least squares. Both the programs use the phi-index method for rainfall separation. In the present study, the Corps of Engineers approach (Subsec. 4.2.2) has been programmed and used whenever there is more than one raingauge station in a basin. Furthermore, the microVAX II computer system of the Centre for Water Resources Engineering and Management, IIT Kanpur has been used for the study and some modifications in the software to suit the microVMS FORTRAN compiler were also made.

4.3. Hydrograph Analysis for a Storm

The input data for the program for estimating the UH parameters and the flood hydrograph by the method of moments for a storm flood event are:

(i) Catchment Area (Km^2);

(ii) Time interval between successive observations of rainfall and runoff (Hours);

(iii) Number of storms to be analysed;

(iv) Number of raingauge stations and the corresponding Theissen weights;

(v) Number of rainfall records for the storm;

(vi) Rainfall depths at specified time interval at each raingauge station for the storm in mm;

(vii) Number of discharge hydrograph ordinates for the storm; and

(viii) Ordinates of the discharge hydrograph for the storm (m^3/sec).

The discharge hydrograph ordinates for each flood event are obtained from

the rating curve that has been prepared for the basin. The rainfall and discharge (stage) data are available at an interval of one hour. The baseflow, assumed to be constant within a storm equal to the initial value, is separated from the observed hydrograph values and the DSRO is obtained. The ERH alongwith the infiltration index is obtained after separating the infiltration losses. These operations are performed by the various subroutines in the two programs. The results include the effective rainfall, the infiltration index and the coefficient of runoff .

For the second computer program based on the method of least squares, initial estimates of the parameters N and K are also required. The program gives the values of the parameters N and K and the UH ordinates for the catchment corresponding to the minimum values of the objective function estimated from the data of the storm precipitation. For a storm-flood event, the Nash model parameters N and K are estimated by the method of moments. Using the values of N and K obtained by the method of moments as initial values, the UH parameters are obtained by the method of least squares. Both the programs compute the ordinates of the discharge hydrograph. The observed and the computed hydrographs are also plotted to see if the flood event is simulated properly. This procedure is repeated for all the flood events identified for the basin. The results are tabulated for further analysis.

To illustrate the results from the analysis, the data and results for a typical storm in Dharsi river basin (Fig. 4.1.) are presented. Dharsi river basin has a catchment area of 91.40 Km^2 and has two raingauge stations. Table 4.1(a) shows the input data for the storm dated 12^{th} to 14^{th} September, 1970. The input data consist of details of the basin and storm, and in particular, rainfall at each station and runoff values. The results of the rainfall separation and DSRO

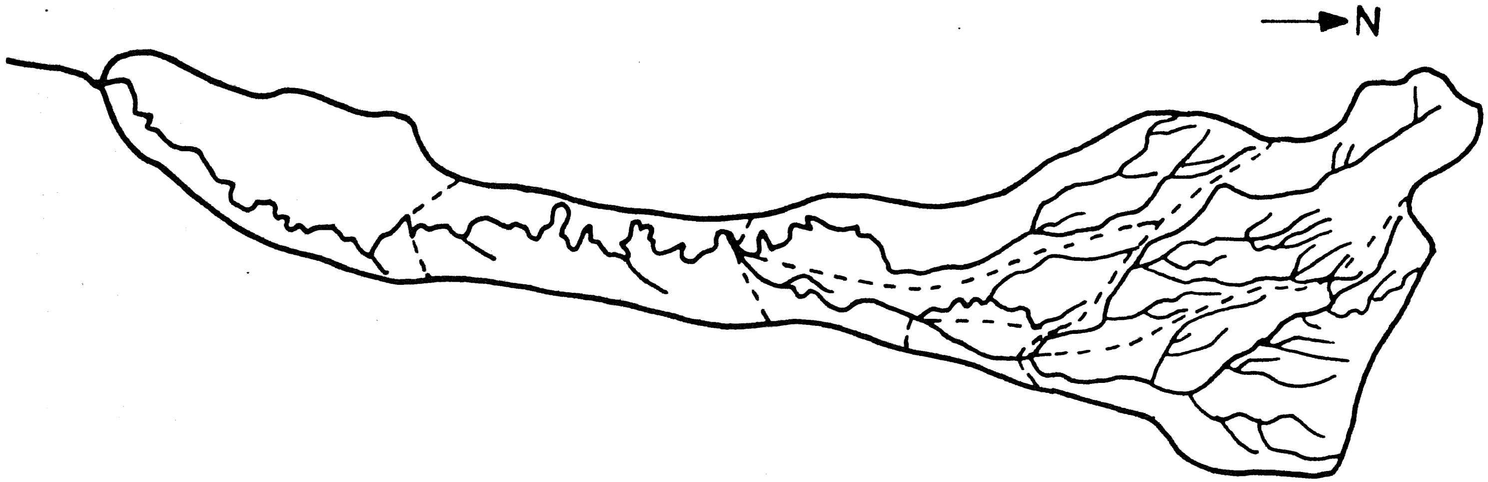


FIG. 4.1 DHARSI RIVER BASIN

also presented in the Table. They include the baseflow, DSRO, weighted rainfall values, infiltration capacity, total rainfall excess and the runoff coefficient as well as the separated rainfall excess values. Table 4.1(b) shows the results of estimation of Nash model parameters by the method of moments and includes the first and second moments of DSRO and ERH respectively, Nash parameters N and K, the IUH and the one-hour UH.

The comparison of observed and computed hydrographs (in Fig.4.2) along with a consolidated list of input data and results of separation analysis are presented in Table 4.1 (c). It also indicates several parameters of the model and the process including observed and computed peaks and times to peak respectively and several measures of error. The results indicate that the average absolute error is around 2%, standard error around 2.6% and the absolute error in peak of about 10.8% which is a little high.

For the analysis using the method of least squares, the input data essentially consist of DSRO and the separated rainfall values derived in a similar manner as that for the method of moments. The initial parameter estimates are the values obtained from the method of moments. The program uses a modified Newton method for minimising the sum of squared errors with a relative tolerance limit of 0.001. The results of analysis are presented in Table 4.2. which is self explanatory. A comparison of the errors indicate that the average standard error and average absolute error are of the order of 1%. The average percentage absolute error is 5.26% and the percentage absolute error in peak is 0.07% and the observed and the computed time to peak are the same. Hence the method of least squares seems to predict the hydrograph better than the method of moments (Fig.4.2).

TABLE 4.1(a) INPUT DATA AND RESULTS OF RAINFALL SEPARATION

UNIT HYDROGRAPH ANALYSIS - NASH MODEL

CATCHMENT DHARSI
DATE 12-14, SEPT, 1970
CATCHMENT AREA(Km^2) 91.40
NO.OF RAINGAUGES 2

THEISSEN WEIGHTS OF EACH RAINGAUGE STATION

0.7700 0.2300

NO OF RAINFALL VALUES= 21

RAINFALL AT EACH STATION (MM)

STATION NO: 1

3.05	0.00	5.08	4.57	2.54	4.57	10.67	25.91	50.80	27.94
29.97	7.72	0.00	0.00	1.27	1.02	0.00	0.51	0.51	0.00
0.00									

STATION NO: 2

6.10	0.00	25.40	5.59	0.00	7.37	0.13	31.00	24.90	6.35
6.10	22.60	0.00	0.00	0.00	0.50	0.00	1.27	3.05	2.29
2.03									

NO. OF RUNOFF VALUES = 37

OBSERVED DISCHARGE HYDROGRAPH (M^3/SEC)

11.75	11.75	11.75	11.75	11.75	11.75	11.75	13.71	15.23	16.99
18.80	20.76	24.30	27.41	29.05	30.75	30.75	30.75	31.63	31.63
30.75	29.05	27.41	25.83	24.30	22.85	22.85	22.14	22.14	22.14
21.44	20.76	19.45	18.80	18.80	18.18	18.18			

WEIGHTED RAINFALL VALUES (MM)

3.75	0.00	9.75	4.80	1.96	5.21	10.09	27.08	44.84	22.97
23.48	11.14	0.00	0.00	0.98	0.90	0.00	0.68	1.09	0.53
0.47									

NO. OF DSRO ORDINATES = 29

BASEFLOW (M^3/SEC)

11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75
11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75
11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75	11.75	

Table 4.1 (a) Contd..

DIRECT SURFACE RUNOFF (M^3/SEC)

3.48	5.24	7.05	9.01	12.55	15.66	17.30	19.00	19.00	19.00
19.88	19.88	19.00	17.30	15.66	14.07	12.55	11.10	11.10	10.39
10.39	10.39	9.69	9.01	7.70	7.05	7.05	6.43	6.43	

TOTAL RAINFALL (MM) 170.74
INFILTRATION CAPACITY (MM/HR) 32.78
TOTAL RAINFALL EXCESS 13.88
RUNOFF COEFFICIENT 0.0813

SEPARATED RAINFALL VALUES (MM)

13.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00							

TABLE 4.1 (b) RESULTS OF ESTIMATION OF NASH MODEL PARAMETERS BY METHOD OF MOMENTS

FIRST MOMENT OF DSRO	13.030
SECOND MOMENT OF DSRO	218.720
FIRST MOMENT OF ERH	0.500
SECOND MOMENT OF ERH	0.250
VALUE OF N	3.200
VALUE OF K	3.915

I.U.H. ORDINATES

0.103	0.365	0.691	1.007	1.275	1.474	1.603	1.666	1.672	1.633
1.560	1.463	1.351	1.232	1.111	0.991	0.878	0.771	0.672	0.583
0.503	0.431	0.368	0.313	0.266	0.224	0.189	0.158	0.132	

SUM OF IUH	0.975
IUH PEAK	1.672
IUH TIME TO PEAK	9 HOURS
AREA OF UH	0.972

UNIT HYDROGRAPH ORDINATES (M^3/SEC)

0.296	0.324	0.515	0.850	1.146	1.380	1.544	1.639	1.673	1.655
1.598	1.513	1.408	1.292	1.171	1.051	0.934	0.823	0.721	0.627
0.542	0.466	0.399	0.340	0.289	0.244	0.206	0.173	0.145	

TABLE 4.1(c) COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS

ORD.NO	RFSTN1	RFSTN2	WTRFALL	EFFRF	BASE FLOW	DSRO	OBSD. DISCH.	COMPUTED DISCHARGE
	MM	MM	MM	MM	M^3/SEC	M^3/SEC	M^3/SEC	M^3/SEC
1	3.05	6.10	3.75	0.00	11.75	0.00	11.75	11.75
2	0.00	0.00	0.00	0.00	11.75	0.00	11.75	11.75
3	5.08	25.40	9.75	0.00	11.75	0.00	11.75	11.75
4	4.57	5.59	4.80	0.00	11.75	0.00	11.75	11.75
5	2.54	0.00	1.96	0.00	11.75	0.00	11.75	11.75
6	4.57	7.37	5.21	0.00	11.75	0.00	11.75	11.75
7	10.67	8.13	10.09	0.00	11.75	0.00	11.75	11.75
8	25.91	31.00	27.08	0.00	11.75	0.00	13.71	11.75
9	50.80	24.90	44.84	13.88	11.75	0.00	15.23	11.75
10	27.94	6.35	22.97	0.00	11.75	4.12	16.99	15.87
11	29.97	6.10	24.48	0.00	11.75	5.33	18.80	17.08
12	7.72	22.60	11.14	0.00	11.75	7.16	20.76	18.91
13	0.00	0.00	0.00	0.00	11.75	11.84	24.30	23.59
14	0.00	0.00	0.00	0.00	11.75	15.95	27.41	27.70
15	1.27	0.00	0.98	0.00	11.75	19.21	29.05	30.96
16	1.02	0.50	0.90	0.00	11.75	21.50	30.75	33.25
17	0.00	0.00	0.00	0.00	11.75	22.82	30.75	34.57
18	0.51	1.27	0.68	0.00	11.75	23.28	30.75	35.04
19	0.51	3.05	1.09	0.00	11.75	23.04	31.63	34.79
20	0.00	2.29	0.53	0.00	11.75	22.25	31.63	34.00
21	0.00	2.03	0.47	0.00	11.75	21.06	30.75	32.81
22	0.00	0.00	0.00	0.00	11.75	19.60	29.05	31.35
23	0.00	0.00	0.00	0.00	11.75	17.98	27.41	29.73
24	0.00	0.00	0.00	0.00	11.75	16.30	25.83	28.05
25	0.00	0.00	0.00	0.00	11.75	14.62	24.30	26.38
26	0.00	0.00	0.00	0.00	11.75	13.00	22.85	24.75
27	0.00	0.00	0.00	0.00	11.75	11.46	22.85	23.21
28	0.00	0.00	0.00	0.00	11.75	10.03	22.14	21.78
29	0.00	0.00	0.00	0.00	11.75	8.73	22.14	20.48
30	0.00	0.00	0.00	0.00	11.75	7.55	22.14	19.30
31	0.00	0.00	0.00	0.00	11.75	6.49	21.44	18.24
32	0.00	0.00	0.00	0.00	11.75	5.56	20.76	17.31
33	0.00	0.00	0.00	0.00	11.75	4.74	19.45	16.49
34	0.00	0.00	0.00	0.00	11.75	4.02	18.80	15.77
35	0.00	0.00	0.00	0.00	11.75	3.40	18.80	15.15
36	0.00	0.00	0.00	0.00	11.75	2.87	18.18	14.62
37	0.00	0.00	0.00	0.00	11.75	2.41	18.18	14.16

UH PEAK (M^3/SEC)	=	1.67
UH TIME TO PEAK (HRS)	=	9.00
EFFICIENCY OF THE MODEL (%)	=	84.10
OBSERVED PEAK (M^3/SEC)	=	31.63
OBSERVED TIME TO PEAK (HRS)	=	19.00
COMPUTED PEAK (M^3/SEC)	=	35.04
COMPUTED TIME TO PEAK (HOURS)	=	18.00
AVERAGE STANDARD ERROR	=	2.63
AVERAGE ABSOLUTE ERROR	=	2.08
AV. PERCENTAGE ABSOLUTE ERROR	=	9.47
PERCENTAGE ABSOLUTE ERROR IN PEAK	=	10.77
PERCENTAGE ABSOLUTE ERROR IN TIME TO PEAK	=	5.26

TABLE 4.2 RESULTS OF ANALYSIS BY NASH MODEL USING METHOD OF LEAST SQUARI

FINAL SOLUTION FOR THE PARAMETERS

N = 3.3339
K = 4.0663

AREA OF UH 0.98417

UNIT HYDROGRAPH ORDINATES

0.269	0.465	0.566	0.664	0.936	1.170	1.351	1.476	1.546	1.567
1.549	1.498	1.423	1.332	1.231	1.125	1.018	0.913	0.813	0.719
0.632	0.552	0.480	0.416	0.358	0.308	0.263	0.224	0.191	0.162
0.137	0.115	0.097	0.081						

ESTIMATED DIRECT SURFACE RUNOFF (M^3/SEC)

0.111	0.152	0.213	0.386	0.544	2.432	3.638	4.919	7.053	9.741
13.761	16.698	18.943	20.455	21.263	21.445	21.104	20.350	19.292	18.026
16.634	15.183	13.727	12.306	10.950	9.678	8.502	7.427	6.457	5.587
4.814	4.132	3.534	3.013						

COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS

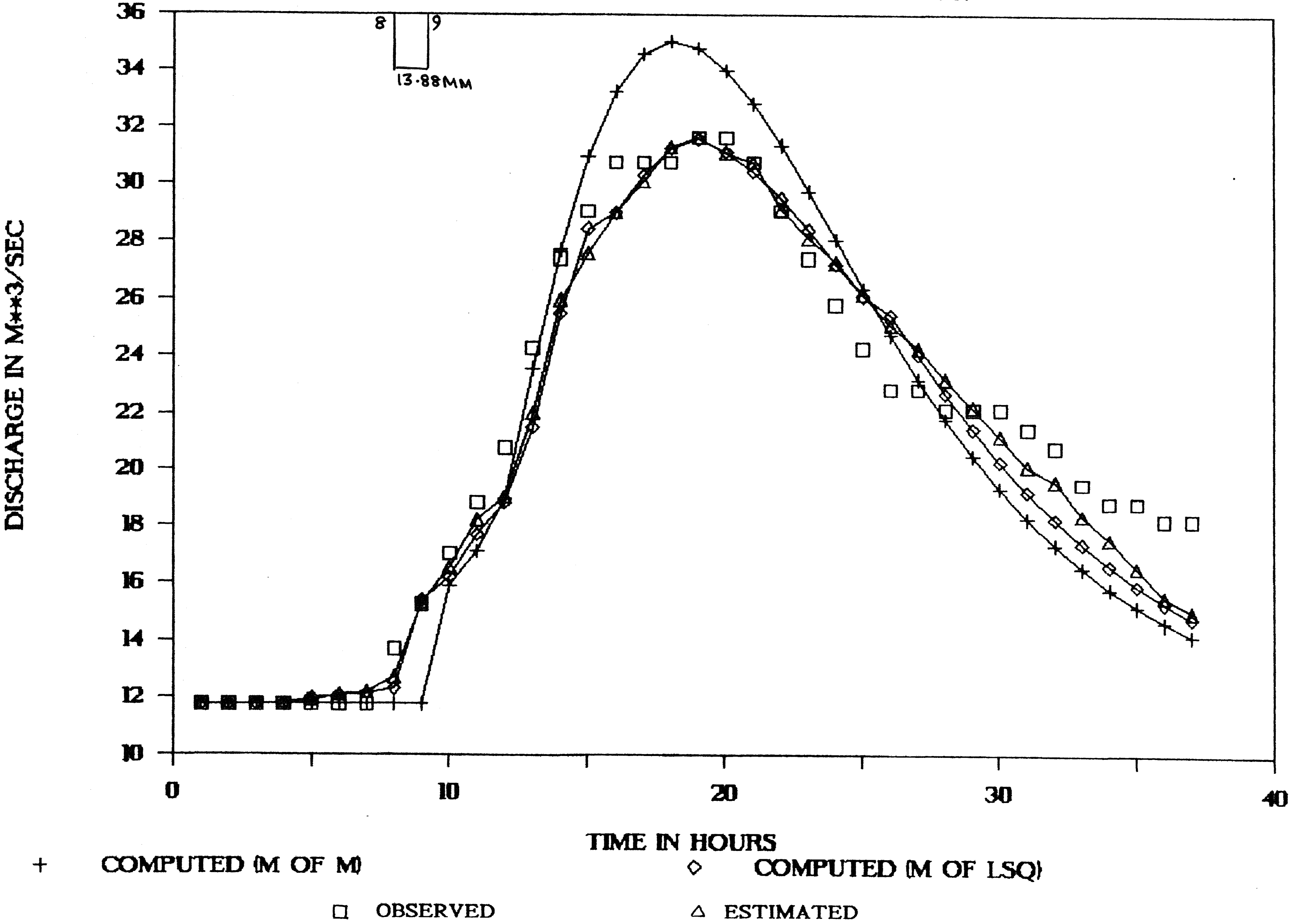
ORD.NO.	OBSERVED DISCHARGE M3/SEC	COMPUTED DISCHARGE M3/SEC
(1)	(2)	(3)
1	11.75	11.75
2	11.75	11.75
3	11.75	11.75
4	11.75	11.75
5	11.75	11.96
6	11.75	12.06
7	11.75	12.14
8	13.71	12.30
9	15.23	15.39
10	16.99	16.18
11	18.80	17.67
12	20.76	18.80
13	24.30	21.49
14	27.41	25.51
15	29.05	28.45
16	30.75	28.99
17	30.75	30.30
18	30.75	31.26
19	31.63	31.58
20	31.63	31.14
21	30.75	30.42
22	29.05	29.48
23	27.41	28.40

TABLE 4.2 Contd...

(1)	(2)	(3)
24	25.83	27.24
25	24.30	26.13
26	22.85	25.48
27	22.85	24.06
28	22.14	22.70
29	22.14	21.43
30	22.14	20.25
31	21.44	19.18
32	20.76	18.21
33	19.45	17.34
34	18.80	16.57
35	18.80	15.88
36	18.18	15.29
37	18.18	14.76

UH PEAK (M^3/SEC) = 1.567
UH TIME TO PEAK (HRS) = 10.00
EFFICIENCY OF THE MODEL = 94.91 %
OBSERVED PEAK (M^3/SEC) = 31.63
OBSERVED TIME TO PEAK (HRS) = 19.00
COMPUTED PEAK (M^3/SEC) = 31.58
COMPUTED TIME TO PEAK (HRS) = 19.00
AVERAGE STANDAR ERROR = 1.542
AVERAGE ABSOLUTE ERROR = 1.006
AV. PERCENTAGE ABSOLUTE ERROR = 5.260
PERCENTAGE ABSOLUTE ERROR IN PEAK = 0.070
PERCENTAGE ABS. ERROR IN TIME TO PEAK0.000

FIG.4.2 DHARSI BASIN — COMPARISON
OF OBSERVED AND COMPUTED HYDROGRAPHS



4.4 Rainfall Runoff Analysis for a Basin

The data for Dharsi basin were available for the years 1968 to 1970 and 18 flood events were identified for analysis. Table 4.3 shows the consolidated results for all the 18 floods in the basin. The first two columns in the Table give the the flood number and the date of occurrence of the event. The next two columns indicate respectively the observed peak discharge value for the flood (m^3/sec) and its time of occurrence. Column 5 shows the total rainfall (mm) for the flood. The next four columns give the results of the separation analysis which include the effective rainfall (mm), baseflow (m^3/sec), infiltration index (mm/hour) and the runoff coefficient.

From the Table, it is observed that the baseflow in this basin varies from 11.75 m^3/sec to 28.20 m^3/sec. There is a wide variation in the values of the infiltration index and the runoff coefficient. A very high runoff coefficient is associated with the highest observed flood peak and it has the lowest infiltration index (2.414 mm/hour) and relatively low rainfall. A very low runoff coefficient is associated with one of the lowest flood peaks and it has the highest infiltration index (33.29 mm/hour) and the highest rainfall (218.4 mm/hour). It is found that the behaviour of the infiltration index and the runoff coefficient with respect to the peak discharge is erratic. This indicates that the floods in this basin are caused by highly variable precipitation and highly variable abstraction.

Columns 10 to 14 represent the results of the Nash model using the method of moments. It is observed that in most cases, the computed peak values are within $\pm$ 10% and generally around 7% or less, except for flood number 11 where there is a large variation.

T A B L E 4.3 RESULTS OF ANALYSIS USING NASH MODEL

DHARSI RIVER BASIN CATCHMENT AREA 91.40 SQ.KM

DATE	QPOBD. M**3/SEC	OB.TIME HRS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEFFT
2	3	4	5	6	7	8	9
260668	39.220	13.000	79.250	12.884	18.180	12.556	0.162
300668	106.760	11.000	88.590	64.009	20.110	2.414	0.757
12130768	44.540	22.000	188.740	20.885	11.750	31.720	0.111
140768	83.170	10.000	66.030	32.497	28.200	7.117	0.492
170768	32.510	11.000	55.370	6.997	21.440	18.913	0.126
20210768	57.850	14.000	119.120	30.892	20.110	14.882	0.259
200968	49.130	10.000	63.000	23.279	18.180	7.227	0.370
290669	52.760	13.000	123.410	18.186	21.096	18.818	0.147
15160869	47.940	12.000	218.440	19.775	14.730	33.292	0.091
20969	75.100	11.000	74.920	32.642	25.060	4.460	0.436
120969	45.680	14.000	71.880	24.502	15.260	9.432	0.341
340670	69.010	13.000	179.830	24.290	20.110	27.460	0.135
450670	102.880	13.000	148.160	47.641	19.454	19.540	0.322
150770	72.010	11.000	124.460	25.335	25.061	23.728	0.204
230770	91.780	14.000	106.940	48.098	20.110	6.087	0.450
50870	37.240	9.000	125.740	12.066	20.110	16.416	0.096
80870	44.660	6.000	91.800	13.780	25.060	14.457	0.150
12140970	31.630	19.000	170.740	13.880	11.750	32.778	0.081

	METHOD OF MOMENTS					METHOD OF LEAST SQUARES					ESTIMATED		
COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COMP.TIME HOURS	N	K	NK	N*	K*	NK*	QPEAK M**3/SEC	TI HOU
11	12	13	14	15	16	17	18	19	20	21	22	23	2
13.000	3.774	3.724	14.054	37.900	14.000	4.000	3.690	14.760	3.327	3.996	13.297	39.210	13.00
10.000	1.490	7.953	11.850	108.730	10.000	1.385	9.265	12.832	1.301	7.779	10.121	106.770	11.00
25.000	3.296	4.534	14.944	39.930	25.000	2.993	5.080	15.204	3.168	4.294	13.603	44.540	23.00
11.000	2.233	4.449	9.935	82.550	10.000	1.840	5.910	10.874	2.009	6.458	12.973	83.180	10.00
10.000	2.992	3.564	10.663	33.450	11.000	3.262	3.776	12.317	3.529	3.621	12.776	32.510	11.00
14.000	2.691	5.541	14.911	60.190	14.000	2.691	5.541	14.911	2.768	5.040	13.952	57.840	14.00
13.000	2.593	5.325	13.808	50.010	13.000	2.593	5.325	13.808	3.030	4.551	13.791	49.120	1.00
17.000	3.883	3.444	13.373	49.890	14.000	3.383	3.938	13.322	2.921	4.755	13.889	52.770	14.00
11.000	2.462	4.308	10.606	48.800	11.000	2.302	4.734	10.898	3.066	4.485	13.749	50.000	13.00
10.000	1.706	5.824	9.936	76.190	11.000	1.650	6.628	10.936	2.251	6.006	13.519	75.800	11.00
16.000	3.023	4.136	12.503	49.750	17.000	2.908	4.925	14.322	3.134	4.358	13.656	46.000	5.00
12.000	2.583	4.269	11.027	61.440	13.000	2.669	4.294	11.461	2.434	5.665	13.786	68.300	3.00
11.000	1.452	6.498	9.435	101.620	11.000	1.605	6.416	10.298	1.418	7.561	10.719	102.580	2.00
9.000	2.079	4.386	9.118	71.520	10.000	2.287	4.528	10.356	2.344	5.833	13.670	72.100	10.00
12.000	1.664	7.131	11.866	82.980	13.000	1.736	7.720	13.402	1.751	6.940	12.149	90.870	13.00
12.000	3.411	3.791	12.931	39.640	12.000	3.411	3.791	12.931	3.387	3.885	13.159	38.130	11.00
14.000	4.294	3.028	13.002	47.030	14.000	4.306	3.062	13.185	3.164	4.301	13.609	44.560	10.00
18.000	3.200	3.915	12.528	31.650	19.000	3.334	4.066	13.556	3.555	3.571	12.696	31.690	19.00

Columns 15 to 18 show the results of the Nash model by the method of least squares. The computed peaks are generally closer to the observed data than those obtained by the method of moments and the error in the peak discharge values are of the order of 3%. However for three floods, viz., 3, 12 and 15, the error in the prediction of peak discharge is larger and perhaps a weighted least squares may lead to a better result. The computed time to peak are often nearly the same as that for observed data except in a few cases where the difference is around two to three hours. However, the deviation in the case of method of least squares is smaller compared to that in the case of method of moments. Hence, for considering the variation of UH parameters and basin characteristics, it is proposed to use the results of the method of least squares.

4.4.1 Parameter variation in a basin

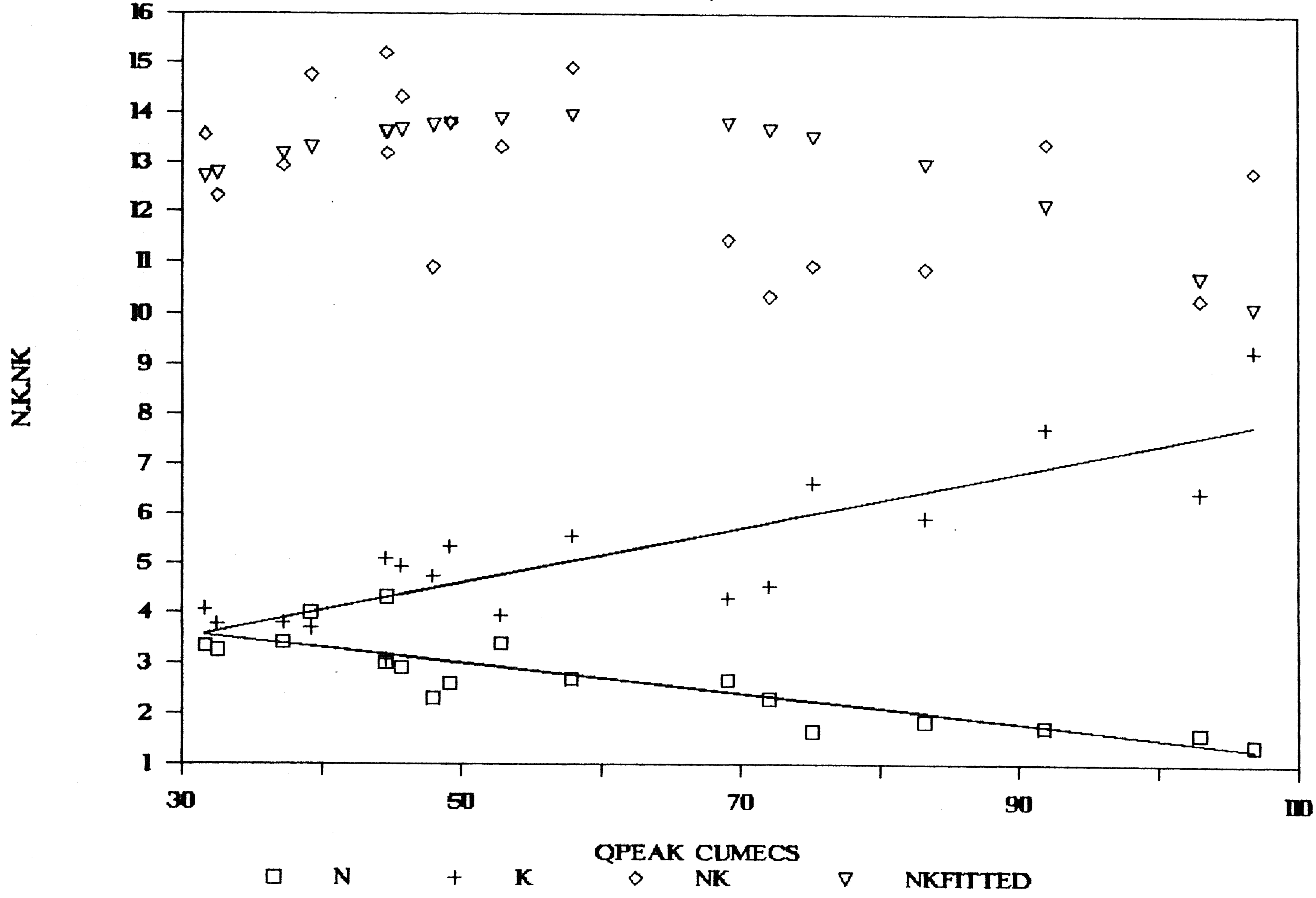
The values of N, K and NK are shown in columns 17 to 19. The values of N, K and NK are plotted against the observed peaks in Fig. 4.3. NK essentially represents the time lag for the storm-flood event and varies from about 10.9 to 15 hours. with a decreasing trend for NK as Q_p increases. This is logical as larger floods will run off quicker. The figure also indicates clearly that the value of N decreases and K increases respectively with the flood peak. A regression line is fitted to these variation by the method of least squares, viz.,

$$N = 4.504 - 0.030\ Q_p; \text{ and} \tag{4.8}$$

$$K = 1.800 + 0.056\ Q_p \tag{4.9}$$

The variation of NK as a function of Q_p is not obvious from the figure. Data errors also have been noted for some storms. The derived equation for the total

DHARSI BASIN - QPEAK VS N,K,NK



time lag in terms of the fitted equation for N and K is given by

$$NK = 8.1072 - 0.1982\ Q_p - 0.00168\ Q_p^2 \tag{4.10}$$

This is also shown in the figure and it seems to fit the observations satisfactorily. The decrease in NK values for smaller Q_ps may be because of a storm occurring near the gauging station producing a larger flood peak (Ramaseshan,1964). The parameter estimates (Eqns. 4.8 to 4.10) are shown in columns 20 to 22 respectively. The estimated Q_p and time to peak for those values are shown in columns 23 and 24 respectively; and for the storm-flood event of Sec.4.3 in Fig.4.2. The fit is considered satisfactory.

Other parameters in the process include coefficient of runoff, infiltration index and baseflow. Attempts were made to correlate these parameters with the storm and flood characteristics. While some general trends were obvious, it is not possible to fit definitive relationships for their variation in a basin especially since the variability about the fitted relationships was very large.

4.5 Comparative Results of Analysis for Different Basins

For each of the basins, rainfall-runoff analysis for all the storms in the basin were carried out. The results are tabulated in Tables 4.4 through 4.10. The plots of observed maximum discharge for each of the floods against the model parameters are also shown in Figs. 4.4 through 4.10.

4.5.1 Routing parameters (Nash Model)

Batjhora basin has a catchment area of 7.33 Km^2 and the results of the analysis of the basin are presented in Table 4.4 and Fig. 4.4. They indicate that the

TABLE 4.4 RESULTS OF ANALYSIS USING NASH MODEL

BATJHORA RIVER BASIN CATCHMENT AREA 7.33 SQ.KM

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEF
1	2	3	4	5	6	7	8	9
1	180764	6.800	6.000	29.718	9.042	2.270	3.342	0.304
2	190764	14.100	3.000	88.138	33.609	2.800	15.708	0.381
3	200764	12.430	3.000	58.928	21.565	3.650	4.449	0.366
4	260764	4.590	5.000	36.068	11.876	1.950	4.842	0.326
5	20964	9.400	4.000	59.944	13.172	2.800	10.677	0.220
6	30964	8.040	9.000	79.248	23.578	2.800	6.670	0.296
7	110964	33.730	5.000	75.946	71.633	3.650	0.479	0.954
8	120964	12.430	7.000	87.630	36.948	3.650	13.337	0.422
9	80865	28.660	6.000	84.836	54.476	5.650	2.915	0.642
10	80965	5.640	12.000	92.710	8.963	1.610	13.884	0.116
11	180965	8.040	3.000	53.594	11.020	2.270	10.000	0.206

. EC	COM.TIME HOURS	METHOD OF MOMENTS N	K	NK	QPCOMP. M**3/SEC	COMP.TIME HOURS	METHOD OF LEAST SQUARES N	K	NK	ESTIMATED N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
40	7.000	1.549	2.184	3.383	6.080	6.000	1.408	2.184	3.075	1.705	2.776	4.734
70	4.000	1.783	1.720	3.067	15.150	4.000	1.401	2.707	3.793	1.508	2.747	4.143
70	3.000	1.346	1.885	2.537	13.170	4.000	2.685	0.867	2.328	1.553	2.753	4.277
10	8.000	2.107	2.715	5.721	4.170	7.000	1.461	4.740	6.925	1.765	2.785	4.915
50	4.000	2.029	1.263	2.563	8.420	4.000	1.480	1.919	2.840	1.635	2.765	4.522
90	8.000	1.486	2.827	4.201	8.020	10.000	1.898	3.324	6.309	1.672	2.771	4.633
10	5.000	0.967	2.799	2.707	33.730	5.000	0.665	3.982	2.648	0.978	2.668	2.610
70	7.000	2.070	2.428	5.026	11.570	7.000	1.701	3.732	6.348	1.553	2.753	4.277
70	7.000	1.197	1.572	1.882	25.760	7.000	1.222	1.645	2.010	1.115	2.688	2.998
10	11.000	1.992	2.866	5.709	4.280	11.000	1.896	3.288	6.234	1.737	2.780	4.829
10	3.000	1.370	1.175	1.610	8.230	3.000	1.073	1.840	1.974	1.672	2.771	4.633

lag time for large floods is smaller. However for small floods, the data are erratic. The regression equation fitted for N, K and NK respectively are considered to be reasonable and are accepted in this study.

Sankhini basin has a catchment area of 7.76 Km^2 and the results for the basin are given Table 4.5 and Fig.4.5. A comparison of results for storms 2 and 5 indicate large inconsistancy of data in that a large flood takes a longer time to travel. The fitted relationships indicate an increase of K with Q_p and a slight increase of the lag time with Q_p. This is not logical. Hence a uniform value of K is adopted for the basin.

Kurti basin has a catchment area of 21.91 Km^2 and the results for this basin are shown in Table 4.6 and Fig.4.6. The results are quite contradictory in that two storms with different effective rainfall each have the same Q_p. Storm 3 has the highest flood peak but also has the largest time lag. Detailed analysis of storm precipitation data indicates two observations of the order of 78mm and 55mm which seem to dominate the process and contribute to effective rainfall while precipitation of the order of 40 mm/hour does not produce any runoff. The high value of phi-index of 41.82 mm/hour for the storm is also unduly large. The results indicate that there are data errors which cause the problem. Similarly the results of the storm indicating the lowest lag time for one of the lowest observed Q_p does not seem to be reliable. Since the data for the basin seem to be inconsistent and erratic, it is proposed not to consider this basin for further analysis.

Mujnai basin has a catchment area of 38.50 Km^2 and has 13 storm flood events. The results of the analysis for the basin are shown in Table 4.7 and Fig.4.7 respectively. Even though there are variations in the results, they are generally

Fig.4.4 BATJHORA BASIN— QPEAK VS N,K,NK

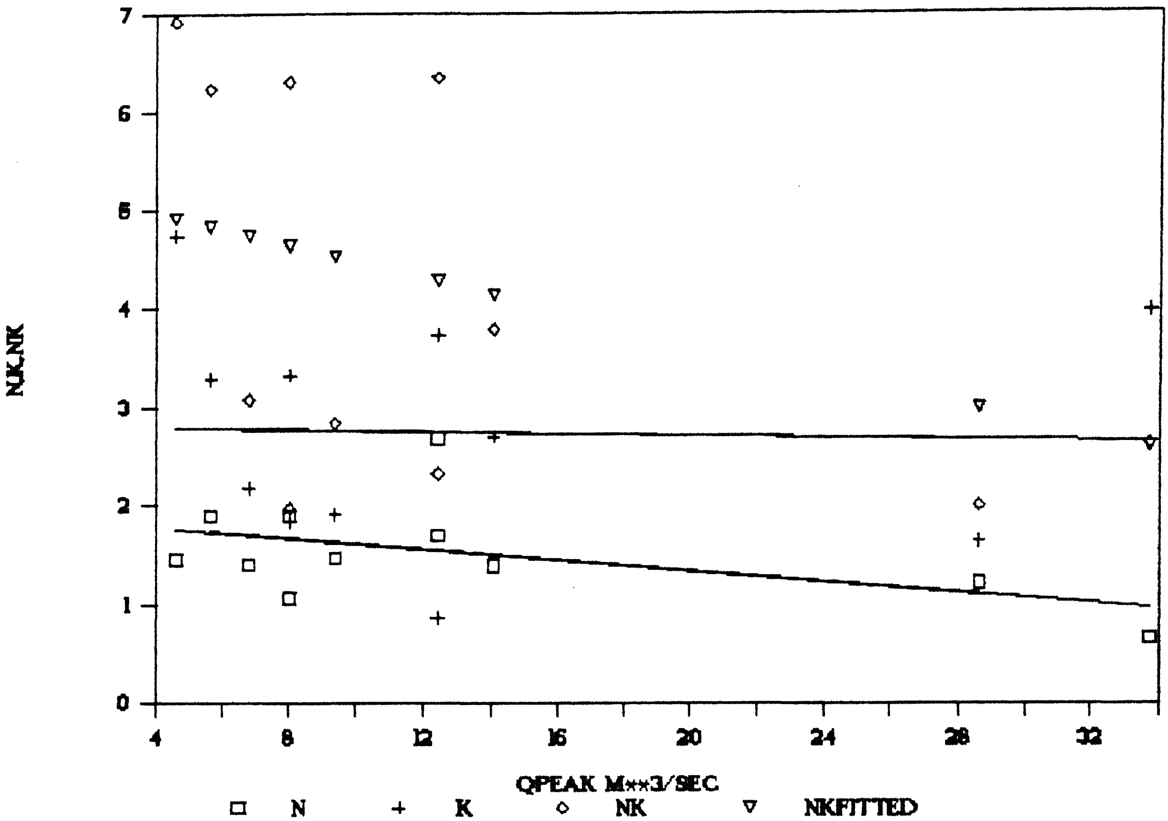
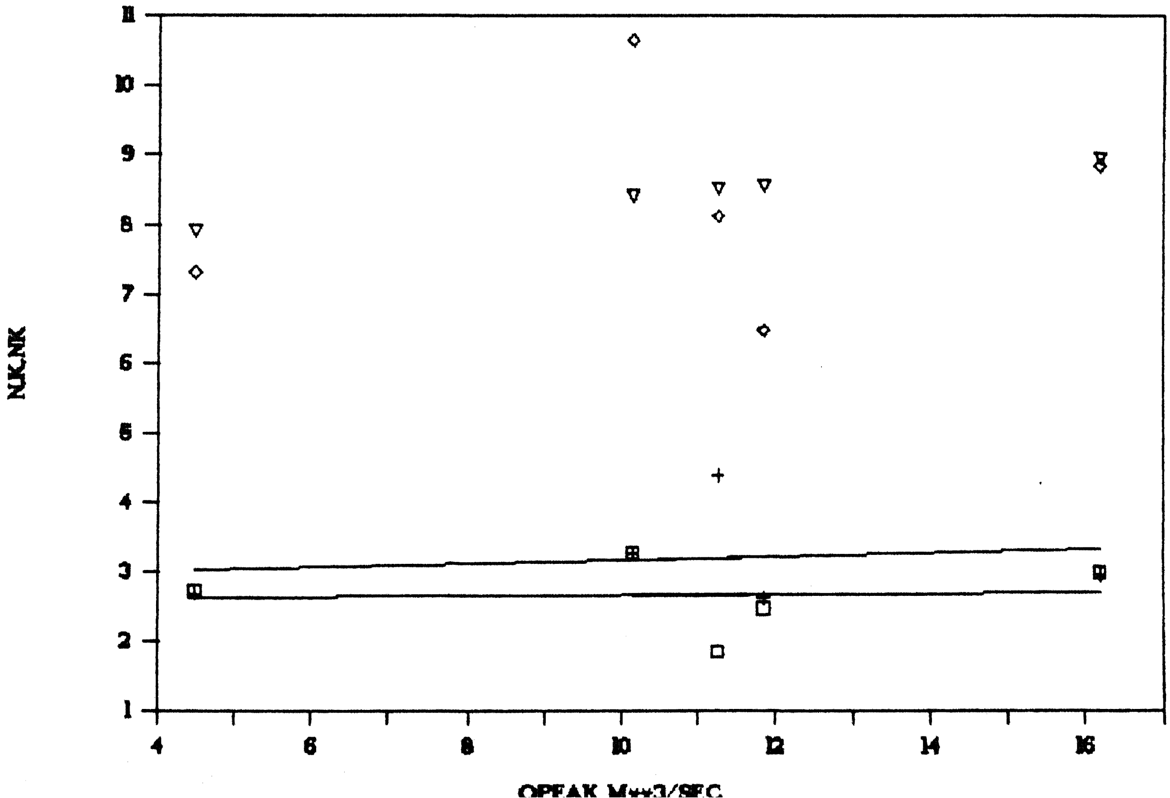


Fig.4.5 SANKHINI BASIN— QPEAK VS N,K,NK



T A B L E 4.5 RESULTS OF ANALYSIS USING NASH MODEL

SANKHINI RIVER BASIN CATCHMENT AREA 7.76 SQ.KM

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEF
1	2	3	4	5	6	7	8	9
1	290764	16.200	6.000	125.730	42.760	6.630	19.528	0.340
2	240765	10.150	14.000	112.776	44.884	2.690	8.140	0.399
3	120865	11.260	12.000	159.766	22.078	6.610	26.680	0.138
4	140865	11.840	15.000	98.552	23.163	7.560	5.216	0.235
5	18190965	4.480	6.500	67.310	16.511	0.730	10.795	0.245

		METHOD OF MOMENTS					METHOD OF LEAST SQUARES			ESTIMATED		
'. EC	COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COMP.TIME HOURS	N	K	NK	N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
30	10.000	3.874	2.296	8.895	14.900	9.000	2.994	2.956	8.850	2.694	3.321	8.948
60	11.000	3.020	2.850	8.607	10.020	13.000	3.278	3.251	10.657	2.658	3.164	8.409
90	10.000	2.330	2.616	6.095	10.850	10.000	1.850	4.390	8.122	2.665	3.193	8.507
40	15.000	2.564	2.334	5.984	12.000	16.000	2.470	2.623	6.479	2.668	3.208	8.559
10	7.000	3.687	1.780	6.563	4.250	7.000	2.723	2.686	7.314	2.624	3.016	7.915

T A B L E 4.6 RESULTS OF ANALYSIS USING NASH MODEL

KURTI RIVER BASIN   CATCHMENT AREA 21.91 SQ.KM.

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEF
1	2	3	4	5	6	7	8	9
1	3010766	30.230	15.000	337.058	60.327	4.930	18.046	0.179
2	100766	30.230	10.000	218.948	44.814	5.870	19.608	0.205
3	20210766	63.550	7.000	312.420	45.916	21.300	41.812	0.147
4	70967	41.810	10.500	114.046	62.483	4.930	6.459	0.548
5	13140768	41.810	14.000	113.538	61.872	8.770	4.446	0.545

		METHOD OF MOMENTS					METHOD OF LEAST SQUARES			ESTIMATED		
QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COMP.TIME HOURS	N	K	NK	N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
37.000	15.000	1.688	3.782	6.384	32.300	15.000	1.306	6.107	7.976	1.227	5.396	6.621
34.750	10.000	1.405	3.446	4.842	34.500	10.000	1.163	4.515	5.251	1.227	5.396	6.621
59.380	9.000	3.394	2.583	8.767	58.500	8.000	2.840	3.072	8.724	2.826	3.163	8.940
51.980	11.000	1.310	3.735	4.893	44.400	11.000	0.946	7.222	6.832	1.783	4.620	8.236
44.240	11.000	2.545	2.067	5.261	42.440	11.000	2.631	2.270	5.973	1.783	4.620	8.236

Fig. 4.6 KURTI BASIN — QPEAK VS N,K,NK

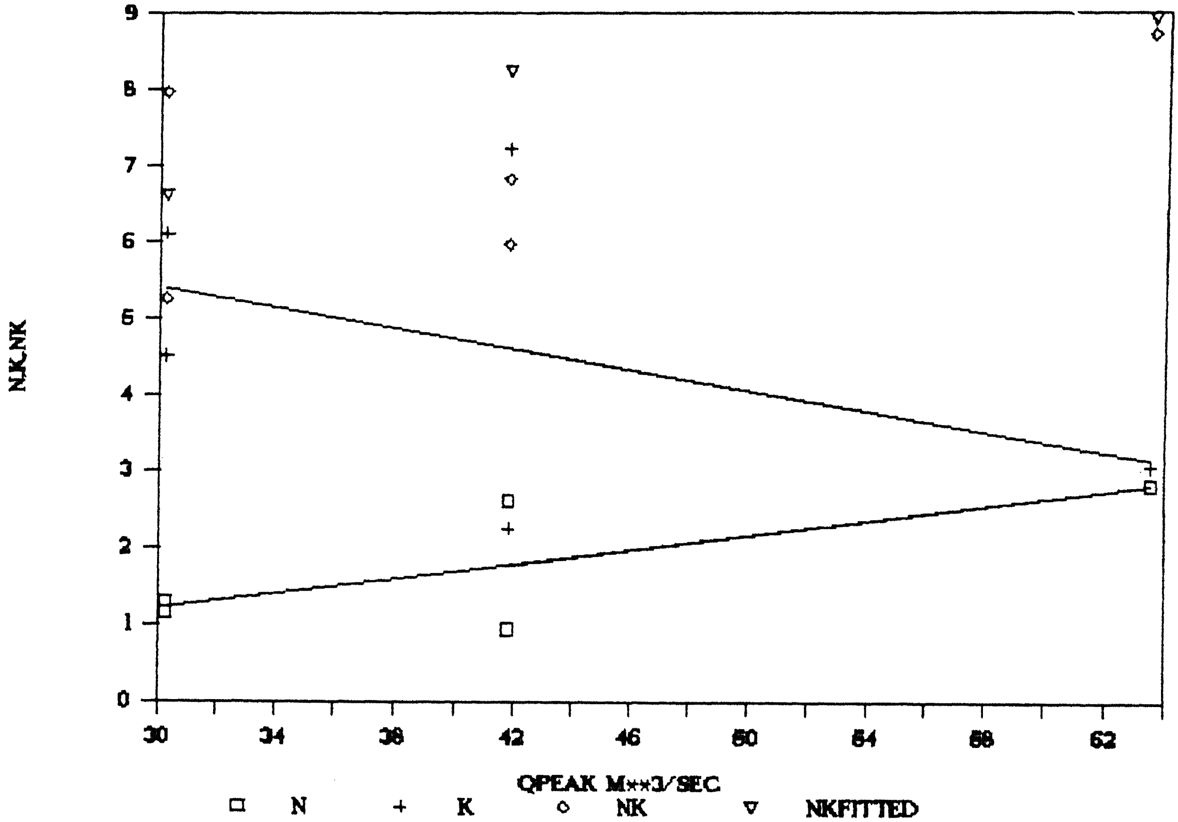
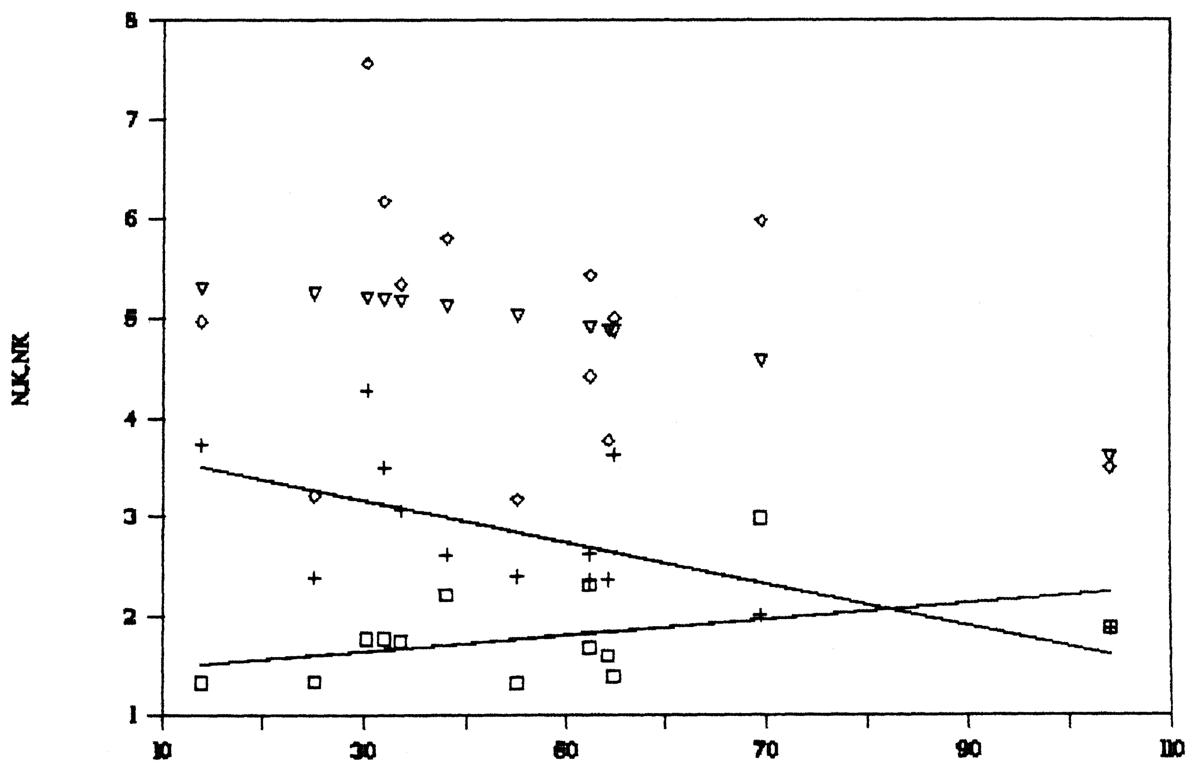


Fig.4.7 MUJNAI BASIN— QPEAK VS N,K,NK



T A B L E 4.7 RESULTS OF ANALYSIS USING NASH MODEL

MUJNAI RIVER BASIN CATCHMENT AREA 38.50 SQ.KM

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEFFT
1	2	3	4	5	6	7	8	9
1	260664	52.440	9.000	81.681	23.396	16.905	7.114	0.286
2	140764	52.440	9.000	135.037	30.318	15.433	12.928	0.225
3	200764	38.060	9.000	122.480	20.767	13.960	12.312	0.170
4	280764	69.550	12.000	162.027	45.578	17.132	11.579	0.281
5	110964	54.280	9.000	96.036	24.096	15.433	6.712	0.251
6	120964	45.110	8.000	72.353	13.687	18.406	8.078	0.189
7	140964	104.150	9.000	125.401	51.980	19.935	8.334	0.415
8	60765	13.900	11.000	85.501	2.380	10.392	10.095	0.028
9	200765	25.170	7.000	46.688	6.085	14.640	7.248	0.130
10	190865	32.000	6.000	38.797	5.126	24.749	7.856	0.132
11	130965	30.270	6.000	83.151	15.945	20.615	7.177	0.192
12	140965	33.670	11.000	85.993	13.668	22.711	7.277	0.159
13	150965	54.820	12.000	88.685	27.379	24.749	6.341	0.309

METHOD OF MOMENTS					METHOD OF LEAST SQUARES							
QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COMP.TIME HOURS	N	K	NK	N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
51.150	9.000	1.826	2.220	4.054	49.090	9.000	1.681	2.629	4.419	1.825	2.691	4.909
52.580	10.000	2.515	2.137	5.375	51.740	9.000	2.312	2.351	5.436	1.825	2.691	4.909
39.910	10.000	2.630	2.105	5.536	38.240	9.000	2.217	2.620	5.809	1.709	2.993	5.116
77.140	11.000	3.279	1.674	5.489	72.450	11.000	2.986	2.005	5.987	1.961	2.331	4.573
52.760	10.000	2.114	1.879	3.972	53.580	9.000	1.592	2.370	3.773	1.839	2.652	4.878
47.950	8.000	1.418	1.930	2.737	45.380	8.000	1.325	2.399	3.179	1.766	2.845	5.023
89.790	9.000	1.675	2.166	3.628	92.150	9.000	1.873	1.863	3.489	2.238	1.605	3.592
16.170	12.000	1.883	1.678	3.160	14.090	12.000	1.332	3.731	4.970	1.516	3.500	5.307
27.650	4.000	1.391	1.740	2.420	25.300	5.000	1.344	2.391	3.214	1.606	3.263	5.242
32.970	5.000	2.382	2.065	4.919	31.040	5.000	1.768	3.498	6.184	1.661	3.120	5.182
33.950	14.000	1.552	3.613	5.607	31.770	15.000	1.772	4.272	7.570	1.647	3.156	5.199
36.340	13.000	2.803	1.406	3.941	34.530	13.000	1.745	3.062	5.343	1.674	3.085	5.165
51.390	13.000	1.509	3.284	4.956	51.490	13.000	1.382	3.624	5.008	1.844	2.641	4.868

consistent and the fitted regression equations appear to be satisfactory.

Talma river basin has a catchment area of 42.10 Km^2 and has 10 rainfall-runoff events (Table 4.8 and Fig.4.8). The largest flood has a large lag time and yet the fitted regression equations are not much affected by it. The relationships are considered satisfactory.

Khaga river basin has a catchment area of 66.00 Km^2 and has 5 storm flood events. The results are shown in Table 4.9 and Fig.4.9. The results are quite consistent and are considered satisfactory.

Mansari river basin is the largest basin considered in the study with a catchment area of 213.00 Km^2. It has 5 storm flood events and the results are shown in Table 4.10 and Fig.4.10. Flood number 1 and 3 have the lowest peaks and have a lag time of 26 hours. Flood number 5 has the highest flood peak and has the lowest NK value of 20.25 hours. These indicate the general trend normally expected for the parameters. Flood number 4 having the second largest flood peak, however, indicates a very large NK value nearly twice that for flood 5 with which it differs in flood peak magnitude by about 3%. This is presumably due to errors, and the parameters of this storm were not used in considering the parameter variation within the basin.

The consolidated list of the relationships between the model parameters and the flood peaks is given in Table 4.11

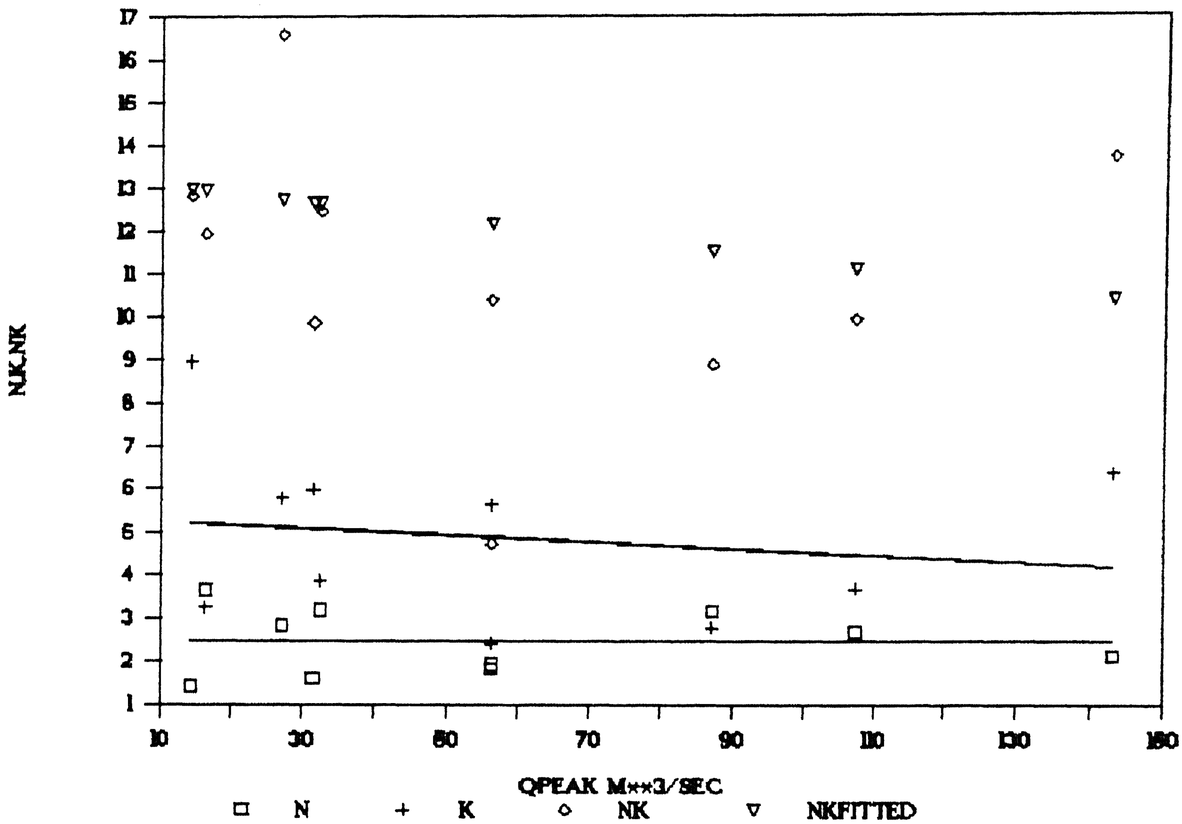
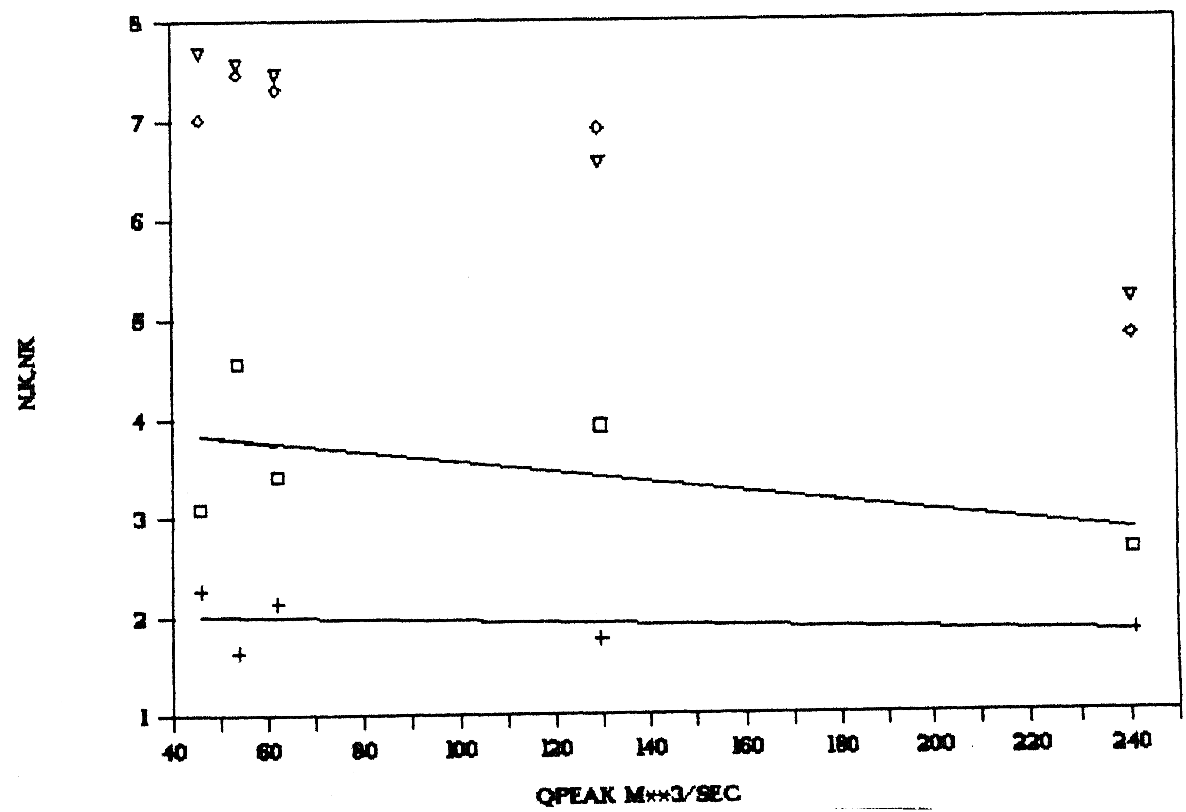


Fig.4.9 KHAGA— QPEAK VS N,K,NK



T A B L E 4.8 RESULTS OF ANALYSIS USING NASH MODEL

TALMA RIVER BASIN CATCHMENT AREA 42.12 SQ.KM

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEFFT
1	2	3	4	5	6	7	8	9
1	18190765	16.340	10.000	32.004	13.389	5.125	5.189	0.418
2	220765	27.240	13.000	85.344	39.194	6.315	5.468	0.459
3	23240765	143.030	13.000	317.754	212.835	10.959	7.876	0.670
4	300765	14.300	17.000	55.118	11.564	6.966	6.390	0.210
5	140865	31.570	11.000	67.056	17.620	17.387	7.754	0.263
6	170767	56.320	12.000	161.290	60.139	7.646	11.442	0.373
7	180767	107.410	11.000	176.276	65.512	51.000	14.227	0.372
8	180968	32.560	14.000	95.504	35.946	11.242	25.715	0.376
9	20210968	87.250	11.000	188.214	64.513	10.704	23.792	0.343
10	12130969	56.320	7.000	55.000	17.651	24.126	11.749	0.321

		METHOD OF MOMENTS					METHOD OF LEAST SQUARES					
QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COMP.TIME HOURS	N	K	NK	N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
17.800	12.000	3.623	2.905	10.525	16.370	13.000	3.661	3.262	11.942	2.485	5.211	12.950
27.900	14.000	2.869	5.793	16.620	27.910	14.000	2.865	5.793	16.597	2.485	5.124	12.733
140.660	16.000	2.525	5.385	13.597	135.690	15.000	2.151	6.386	13.736	2.485	4.198	10.431
15.640	16.000	1.481	7.127	10.555	14.300	16.000	1.431	8.959	12.820	2.485	5.228	12.991
34.450	11.000	1.945	4.293	8.350	31.820	11.000	1.641	6.000	9.846	2.485	5.089	12.647
58.920	12.000	1.983	4.981	9.877	56.130	12.000	1.843	5.634	10.383	2.485	4.891	12.155
119.140	11.000	3.297	2.602	8.579	107.920	11.000	2.688	3.701	9.948	2.485	4.483	11.140
40.510	12.000	3.347	3.584	11.996	39.090	12.000	3.206	3.884	12.452	2.485	5.082	12.628
69.240	10.000	2.300	4.129	9.497	78.380	11.000	3.172	2.803	8.891	2.485	4.644	11.540
63.730	7.000	2.445	1.614	3.946	55.780	7.000	1.941	2.438	4.732	2.485	4.891	12.155

T A B L E 4.9 RESULTS OF ANALYSIS USING NASH MODEL

KHAGA RIVER BASIN   CATCHMENT AREA 66.00 SQ.KM.

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEF
1	2	3	4	5	6	7	8	9
1	150781	241.020	8.000	132.250	97.857	16.590	3.558	0.724
2	240881	129.500	9.000	149.750	39.815	12.000	17.931	0.266
3	280784	45.930	8.000	58.400	15.327	4.430	7.550	0.262
4	140984	62.130	14.000	161.500	39.225	5.510	12.983	0.243
5	160984	53.900	9.500	55.100	16.740	16.590	5.749	0.304

		METHOD OF MOMENTS					METHOD OF LEAST SQUARES			ESTIMATED		
QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
242.650	8.000	2.922	1.603	4.684	234.800	8.000	2.637	1.820	4.799	2.847	1.817	5.173
126.710	9.000	3.829	1.643	6.291	118.400	8.000	3.910	1.768	6.913	3.404	1.928	6.566
34.440	10.000	3.488	2.172	7.576	35.600	9.000	3.093	2.269	7.018	3.822	2.012	7.691
64.000	14.000	3.192	2.568	8.197	64.850	14.000	3.421	2.140	7.321	3.741	1.996	7.467
57.500	9.000	5.580	1.248	6.964	52.500	9.000	4.557	1.640	7.473	3.782	2.004	7.581

T A B L E 4.10 RESULTS OF ANALYSIS USING NASH MODEL

MANSARI RIVER BASIN CATCHMENT AREA 213.00 SQ.KM

	DATE	QPOBD. M**3/SEC	OB.TIME HOURS	RFALL MM	EFFRF MM	BASEFLO M**3/SEC	PHIINDEX MM/HOUR	R-O COEF
1	2	3	4	5	6	7	8	9
1	80670	25.510	32.000	81.502	12.760	2.095	16.330	0.157
2	26290970	35.930	23.000	90.921	13.231	11.214	17.761	0.146
3	300771	20.730	22.000	64.510	7.595	5.918	11.597	0.118
4	300971	59.210	23.000	32.600	15.993	13.706	4.488	0.491
5	15190871	57.400	25.000	158.671	29.476	17.359	10.112	0.186

METHOD OF MOMENTS					METHOD OF LEAST SQUARES					ESTIMATED		
QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	QPCOMP. M**3/SEC	COM.TIME HOURS	N	K	NK	N*	K*	NK*
10	11	12	13	14	15	16	17	18	19	20	21	22
27.220	30.000	5.491	5.514	30.277	27.600	25.000	4.011	6.575	26.372	3.998	6.826	27.291
40.570	26.000	5.558	4.792	26.634	34.140	26.000	4.002	7.550	30.215	3.998	6.399	25.583
20.990	28.000	4.763	5.993	28.545	21.240	25.000	4.004	6.563	26.278	3.998	7.022	28.074
58.120	23.000	4.440	4.243	18.839	56.960	24.000	4.018	5.041	20.255	3.998	5.444	21.767
57.320	32.000	3.197	10.696	34.195	58.730	30.000	2.999	10.737	32.200	3.563	4.225	15.054

Fig.4.10 MANSARI BASIN— QPEAK VS N,K,NK

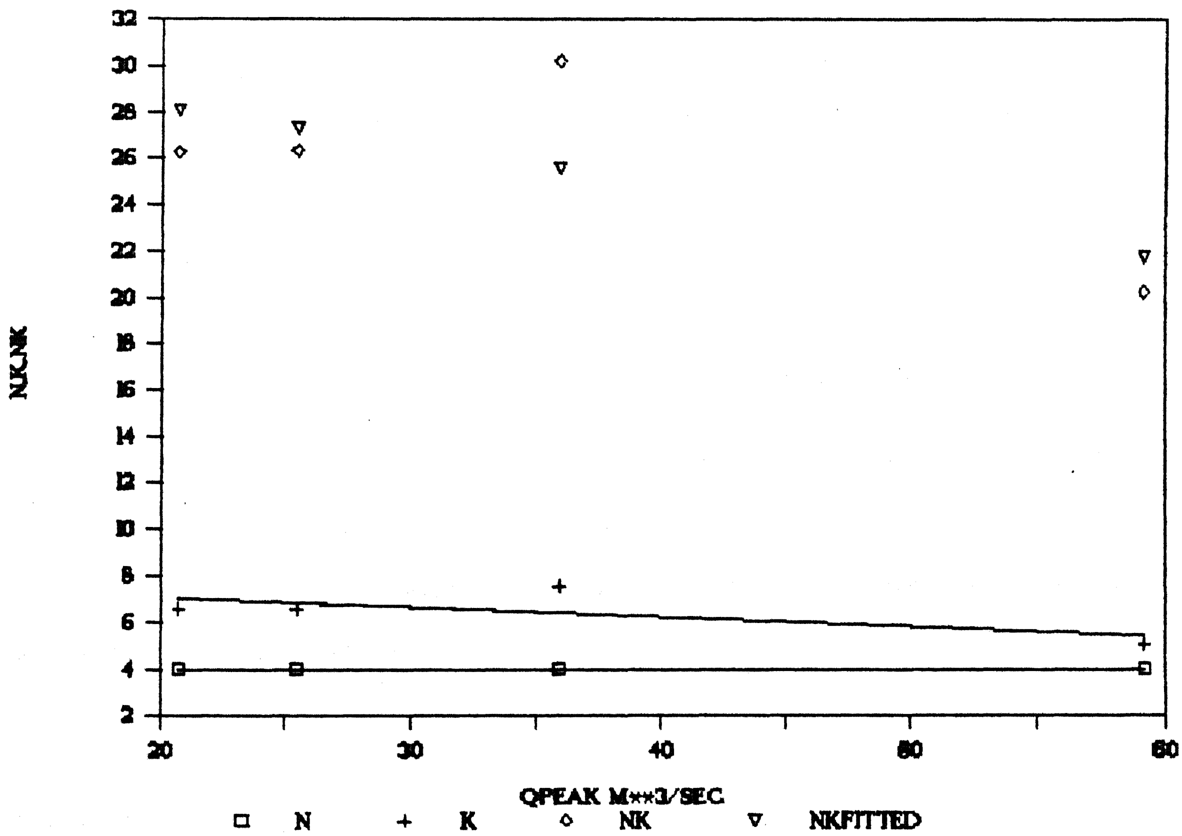


Table 4.11 Regression relationships between model parameters and Q_p

Basin	Regression Relationsips	
Batjhora	$N = 1.889 - 0.027\ Q_p$	$K = 2.803 - 0.004\ Q_p$
Sankhini	$N = 2.597 + 0.006\ Q_p$	$K = 2.900 + 0.026\ Q_p$
Mujnai	$N = 1.405 + 0.008\ Q_p$	$K = 3.792 - 0.021\ Q_p$
Talma	$N = 2.485$	$K = 5.342 - 0.008\ Q_p$
Khaga	$N = 4.502 - 0.005\ Q_p$	$K = 2.053 - 0.001\ Q_p$
Dharsi	$N = 4.504 - 0.030\ Q_p$	$K = 1.800 + 0.056\ Q_p$
Mansari	$N = 3.998$	$K = 7.872 - 0.042\ Q_p$

4.5.2 Storm runoff parameters

Perusal of Tables 4.3 to 4.10 show that the infiltration index, the coefficient of runoff and the baseflow vary from storm to storm in each basin and is often erratic. Attempts were made to correlate them with storm and runoff parameters; but they were unsuccessful. This indicates the inherent variability of the process. However, the Tables indicate that the baseflow/unit area may be a better parameter than the baseflow itself.

4.6 Regionalisation of Rainfall-Runoff Parameters

From the analysis of the storm flood events in the various basins, parameters are to be identified as representative of the region under study for use in flood estimation for the catchments in the region. Table 4.12 show the ranges of these parameters for the various basins.

Table 4.12 Ranges for storm runoff parameters

Sl.No.	Basin	Ranges For Infiltration index mm/hour	Baseflow per unit area $m^3/sec/Km^2$	Coeff. of Runoff
1.	Batjhora	0.50 - 15.70	0.22 - 0.77	0.12 - 0.95
2.	Sankhini	5.00 - 26.70	0.94 - 0.97	0.14 - 0.40
3.	Mujnai	7.00 - 13.00	0.27 - 0.64	0.13 - 0.42
4.	Talma	5.00 - 26.00	0.12 - 1.20	0.21 - 0.67
5.	Khaga	3.50 - 18.00	0.07 - 0.25	0.24 - 0.72
6.	Dharsi	2.40 - 33.00	0.13 - 0.31	0.10 - 0.76
7.	Mansari	4.50 - 17.80	0.01 - 0.08	0.12 - 0.49

As seen from the Table, the coefficient of runoff varies from 0.1 to 0.95. Since variation is too large and no trend is observed for its variation, this parameter cannot be used for regionalisation. The infiltration rate in the basin is another parameter that could be considered for regionalisation and for the design flood a minimum value of the infiltration rate which will lead to a large flood can be taken as a design parameter. 5.00 mm/hour seems to be the minimum value for the basins in general. This value corresponds to a silty loam soil or sandy clay loam soil which is considered realistic. So this value can be taken for computation of design flood for the catchments in the region. This value can be considered to be the general minimum value for the phi-index for all the basins, particularly during the intensive storms generally met with in the basin. Accordingly an abstraction index of 5mm/hour is adopted for all the basins in the study. .

The baseflow generally increases with basin area. The range of variation of baseflow/unit area for each of the basins is also shown in Table 4.12. An enveloping

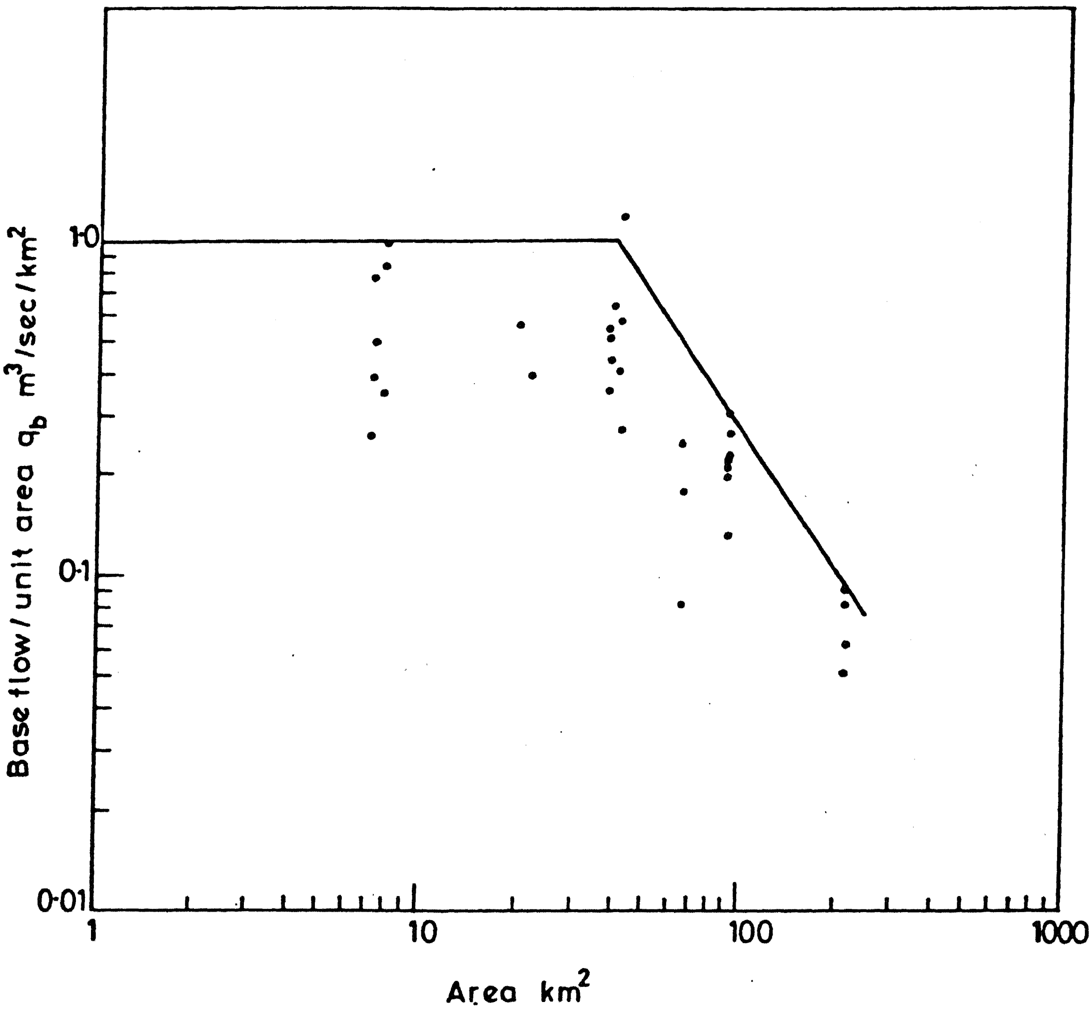
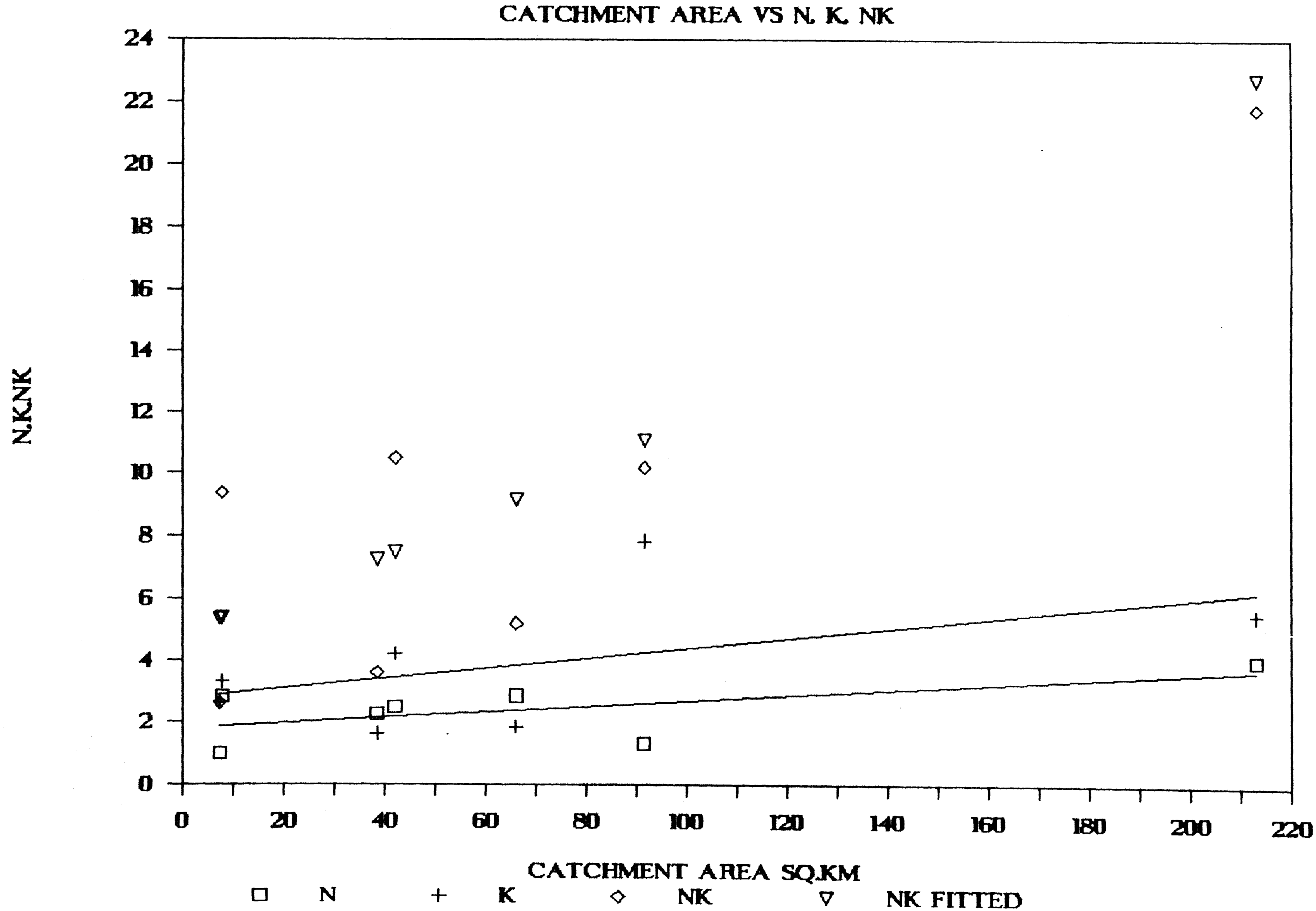


FIG. 4·11 ENVELOPING CURVE FOR BASE FLOW

Fig.4.12 NORTH BRAHMAPUTRA SUBZONE



iii) Estimate the effective rainfall for the storm assuming the minimum phi-index of 5.00 mm/hour;

iv) Estimate the Nash model parameter for the basin using Eqns.4.10 and 4.11;

v) Calculate the IUH ordinates in mm/hour from Eqn. 4.1;

vi) Estimate the 1-hour UH in mm/hour;

vii) Convert the UH values to m^3/sec;

viii) Calculate the DSRO ordinates by convolving the 1 hour effective rainfall values with the UH values;

ix) Estimate the baseflow from Fig. 4.12; and

x) Calculate the flood discharge values by adding the baseflow to the DSRO ordinates.

To demonstrate the use of the model developed in this Chapter, the Nash model is applied to Kurti basin in the region, the data for which were not used in the study.

4.7.1 Design storm

For Kurti basin, Kirpich equation (Eqn.2.3) is used to estimate the time of concentration (t_c) with L = 14.81 Km. and S = 0.03166 as equal to 118.63 minutes, say, 2 hours.

Depth-duration-frequency (DDF) maps developed by IMD (1974) for 50 year return period are used to estimate the design storm. The basin is located in the maps and the rainfall for various durations of 30 minutes, 1 hour, 3 hours,.....etc., upto 24 hours were noted from the respective DDF maps. Point rainfall depths for 50 years frequency were plotted as a function of duration on a log-log paper (Fig.4.13). The 2 hour precipitation value is interpolated as 105 mm. The point

rainfall values are converted to areal rainfall values by multiplication with area reduction factors (CWC, 1973). The hourly storm values are then derived. CWC/IMD adopts a sequencing procedure for the design storm precipitation, viz., the highest hourly precipitation is kept in the middle, the next highest value follows it, the third precedes it and the others alternate. Since the design hyetograph has a duration of 2 hours only, it has hourly precipitations of 67.22 mm and 19.57 mm (Table 4.13).

Table 4.13 Calculation of Design Storm Hyetograph

Duration	Point Rainfall from DDF Curve mm	Areal to Point Rainfall Ratio	Areal Rainfall mm	Hourly Rainfall of Design Storm mm	Abstrac--tion mm/hour	Effective Rainfall mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)
30 min.	65	68.33	46.46	-	-	-
1 hour	88	76.39	67.22	67.22	5.00	62.22
2 hours	105	82.66	86.79	19.57	5.00	14.57

4.7.2 Design unit hydrograph

The Kurti basin has a catchment area of 21.91 Km^2. Using Eqns.4.10 and 4.11, the values of the Nash model parameters for the basin are found to be N = 1.9855; and K = 3.1170. Table 4.14 shows the estimation of design flood for the basin. The IUH ordinates at hourly intervals are calculated from Eqn.4.1. The 1 hour UH ordinates are estimated by integration of 1 minute IUH values. Since the area of the basin is 21.91 Km^2, the corresponding values in m^3/sec are shown in column 3. The effective rainfall values for the design storm are convolved with the 1 hour UH values to yield DSRO ordinates (column 4). The baseflow of 21.91 m^3/sec is

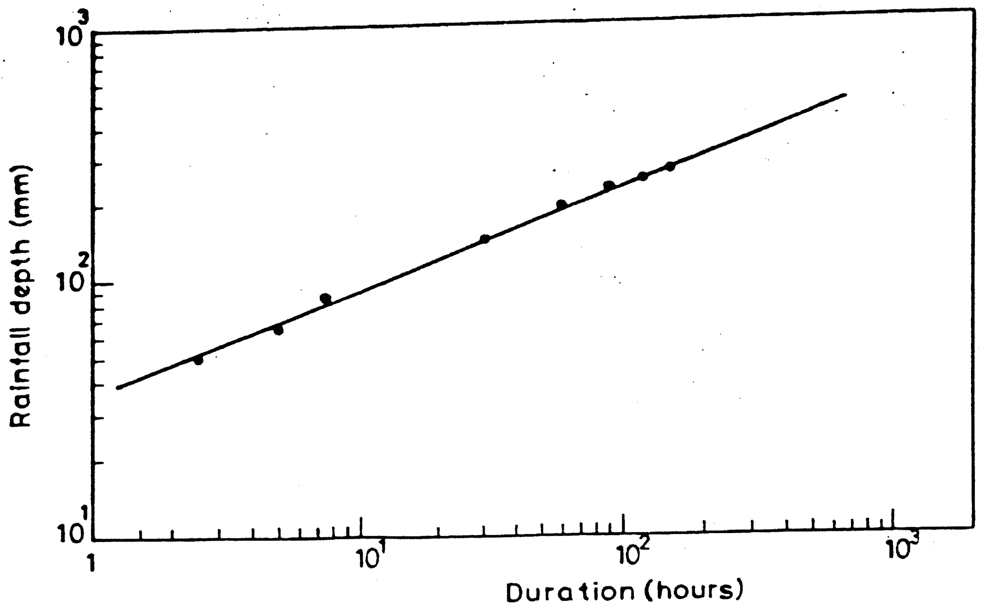


Fig.4.13 Depth duration curve for 50 year return period for the region

TABLE 4.14 ESTIMATION OF DESIGN FLOOD FOR KURTI BASIN

N = 1.9855 K = 3.1170 AREA = 21.91 SQ.KM
BASEFLOW = 21.91 M**3/SEC DT = 1 HOUR

EFFECTIVE RAINFALL MM	1 HOUR UNIT HYDROGRAPH ORDINATES (M**3/SEC)	DSRO (M**3/SEC)	DISCHARGE HYDROGRAPH ORDINATES (M**3/SEC)	TIME HOURS	EFFECTIVE RAINFALL MM	1 HOUR UNIT HYDROGRAPH ORDINATES (M**3/SEC)	DSRO (M**3/SEC)	DISCHARGE HYDROGRAPH ORDINATES (M**3/SEC)
(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
	0.0000	0.0000	21.9100	17		0.0516	3.9620	25.8720
62.22	0.2323	17.8315	39.7415	18		0.0396	3.0369	24.9469
14.57	0.5661	43.4587	65.3687	19		0.0337	2.5884	24.4984
	0.6950	53.3568	75.2668	20		0.0266	2.0441	23.9541
	0.7093	54.4547	76.3647	21		0.0175	1.3468	23.2568
	0.6628	50.8805	72.7905	22		0.0133	1.0197	22.9297
	0.5879	45.1337	67.0437	23		0.0102	0.7826	22.6926
	0.5042	38.7070	60.6170	24		0.0077	0.5910	22.5010
	0.4220	32.3995	54.3095	25		0.0058	0.4478	22.3578
	0.3509	26.9424	48.8524	26		0.0044	0.3397	22.2497
	0.2852	21.8917	43.8017	27		0.0033	0.2539	22.1639
	0.2251	17.2825	39.1925	28		0.0025	0.1892	22.0992
	0.1786	13.7130	35.6230	29		0.0019	0.1437	22.0537
	0.1408	10.8092	32.7192	30		0.0014	0.1082	22.0182
	0.1140	8.7511	30.6611	31		0.0009	0.0706	21.9806
	0.0897	6.8892	28.7992	32		0.0003	0.0243	21.9343
	0.0669	5.1394	27.0494					

estimated for the basin from Fig.4.12. Using this value, the flood discharge values are computed and are shown in column 5. The flood peak of 76.36 m^3/sec occurs at 4 hours for this basin.

For small basins, it is desirable to derive the design storm at shorter intervals, say 15 to 30 minutes and use it to derive the design flood. The design flood for 15 minute interval has been derived and is shown in Table 4.14a. The results indicate that the hydrograph peak is changed marginally (from 76.36 m^3/sec to 77.21 m^3/sec) but its time of occurrence is earlier by 45 minutes.

4.7.3 Comparison of results with Snyder's approach

Snyder's approach (Eqns.2.12 to 2.15) requires the values of the coefficients C_t and C_p. C_t and C_p are computed for a gauged watershed nearby the ungauged one. In the present study, the coefficients are computed for the Mujnai basin. From Eqns. 4.10 and 4.11, the Nash model parameters for this basin are calculated as N=2.131 and K=3.382 . The 1 hour UH ordinates are obtained (Table 4.15). From the catchment map of this basin, the values of L, L_c and S are obtained as 21.33 Km, 11.75 Km and 0.03163 respectively. The peak discharge of 1 hour UH for the basin is 1.0863 m^3/sec and it occurs at t = 4 hours. Using these values, the Snyder's coefficients C_t and C_3 are calculated. For the Mujnai basin, t_R=1 hour; t_{pR}=3.5hours. Hence t_r = 13/21 hours; t_p = 71.5/21 hours; q_{pR} = 0.02822 m^3/sec/Km^2. Thus C_t = 0.888 and C_p = 0.0359. Using the above values of C_t and C_p for the Kurti basin, t_R = 1 hour; t_{pR} = 2.98 hours; q_{pR} = 0.0331 m^3/sec/Km^2 and Q_{pR} = 0.726 m^3/sec compares well with t_{pR} = 3.5 hors and Q_{pR} = 0.709 m^3/sec derived using the UH approach in the study.

N = 1.985K = 3.117 AREA = 21.91 SQ.KM
BASEFLOW = 21.91 M**3/SEC DT = 15 MIN

Time HOURS	EFFECTIVE RAINFALL	UH ORDINATES	DSRO	DISCH. HYDROGRAPH ORDINATES
	MM	M**3/SEC	M**3/SEC	M**3/SEC
(1)	(2)	(3)	(4)	(5)
0.0000		0.0000	0.0000	21.9100
0.2500	30.25	0.0753	5.7758	27.6858
0.5000	11.67	0.2128	16.3305	38.2405
0.7500	10.54	0.3269	25.0808	46.9908
1.0000	9.76	0.4213	32.3271	54.2371
1.2500	4.81	0.4988	38.2709	60.1809
1.5000	3.35	0.5615	43.0807	64.9907
1.7500	3.22	0.6113	46.9026	68.8126
2.0000	3.13	0.6499	49.8646	71.7746
2.2500		0.6787	52.0795	73.9895
2.5000		0.6992	53.6472	75.5572
2.7500		0.7123	54.6560	76.5660
3.0000		0.7192	55.1839	77.0939
3.2500		0.7207	55.3003	77.2103
3.5000		0.7177	55.0663	76.9763
3.7500		0.7107	54.5358	76.4458
4.0000		0.7006	53.7566	75.6666
4.2500		0.6877	52.7705	74.6805
4.5000		0.6727	51.6145	73.5245
4.7500		0.6558	50.3211	72.2311
5.0000		0.6375	48.9184	70.8284
5.2500		0.6182	47.4314	69.3414
5.5000		0.5980	45.8815	67.7915
5.7500		0.5772	44.2874	66.1974
6.0000		0.5560	42.6654	64.5754
6.2500		0.5347	41.0293	62.9393
6.5000		0.5134	39.3910	61.3010
6.7500		0.4921	37.7607	59.6707
7.0000		0.4711	36.1471	58.0571
7.2500		0.4504	34.5572	56.4672
7.5000		0.4300	32.9971	54.9071
7.7500		0.4102	31.4718	53.3818
8.0000		0.3908	29.9850	51.8950
8.2500		0.3720	28.5402	50.4502
8.5000		0.3537	27.1395	49.0495
8.7500		0.3360	25.7849	47.6949
9.0000		0.3190	24.4776	46.3876
9.2500		0.3026	23.2184	45.1284
9.5000		0.2868	22.0077	43.9177
9.7500		0.2717	20.8455	42.7555
0.0000		0.2572	19.7315	41.6415
0.2500		0.2433	18.6652	40.5752
0.5000		0.2300	17.6459	39.5559
0.7500		0.2173	16.6728	38.5828
1.0000		0.2052	15.7447	37.6547
1.2500		0.1937	14.8605	36.7705

Time HOURS	EFFECTIVE RAINFALL	UH ORDINATES	DSRO	DISCH. HYDROGRAPH ORDINATES
	MM	M**3/SEC	M**3/SEC	M**3/SEC
(1)	(2)	(3)	(4)	(5)
11.5000		0.1827	14.0190	35.9290
11.7500		0.1723	13.2188	35.1288
12.0000		0.1624	12.4586	34.3686
12.2500		0.1530	11.7370	33.6470
12.5000		0.1440	11.0526	32.9626
12.7500		0.1356	10.4039	32.3139
13.0000		0.1276	9.7894	31.6994
13.2500		0.1200	9.2079	31.1179
13.5000		0.1128	8.6577	30.5677
13.7500		0.1061	8.1377	30.0477
14.0000		0.0997	7.6463	29.5563
14.2500		0.0936	7.1823	29.0923
14.5000		0.0879	6.7444	28.6544
14.7500		0.0825	6.3313	28.2413
15.0000		0.0774	5.9418	27.8518
15.2500		0.0727	5.5747	27.4847
15.5000		0.0681	5.2289	27.1389
15.7500		0.0639	4.9032	26.8132
16.0000		0.0599	4.5967	26.5067
16.2500		0.0561	4.3082	26.2182
16.5000		0.0526	4.0370	25.9470
16.7500		0.0493	3.7819	25.6919
17.0000		0.0462	3.5421	25.4521
17.2500		0.0432	3.3169	25.2269
17.5000		0.0405	3.1053	25.0153
17.7500		0.0379	2.9066	24.8166
18.0000		0.0355	2.7201	24.6301
18.2500		0.0332	2.5450	24.4550
18.5000		0.0310	2.3808	24.2908
18.7500		0.0290	2.2268	24.1368
19.0000		0.0271	2.0824	23.9924
19.2500		0.0254	1.9470	23.8570
19.5000		0.0237	1.8200	23.7300
19.7500		0.0222	1.7011	23.6111
20.0000		0.0207	1.5897	23.4997
20.2500		0.0194	1.4854	23.3954
20.5000		0.0181	1.3877	23.2977
20.7500		0.0169	1.2962	23.2062
21.0000		0.0158	1.2106	23.1206
21.2500		0.0147	1.1305	23.0405
21.5000		0.0138	1.0555	22.9655
21.7500		0.0128	0.9854	22.8954
22.0000		0.0120	0.9198	22.8298
22.2500		0.0112	0.8585	22.7685
22.5000		0.0104	0.8012	22.7112

Time HOURS	EFFECTIVE RAINFALL	UH ORDINATES	DSRO	DISCH. HYDROGRAPH ORDINATES
	MM	M**3/SEC	M**3/SEC	M**3/SEC
(1)	(2)	(3)	(4)	(5)
22.7500		0.0097	0.7476	22.6576
23.0000		0.0091	0.6975	22.6075
23.2500		0.0085	0.6506	22.5606
23.5000		0.0079	0.6069	22.5169
23.7500		0.0074	0.5660	22.4760
24.0000		0.0069	0.5278	22.4378
24.2500		0.0064	0.4922	22.4022
24.5000		0.0060	0.4589	22.3689
24.7500		0.0056	0.4278	22.3378
25.0000		0.0052	0.3988	22.3088
25.2500		0.0048	0.3717	22.2817
25.5000		0.0045	0.3464	22.2564
25.7500		0.0042	0.3228	22.2328
26.0000		0.0039	0.3008	22.2108
26.2500		0.0037	0.2803	22.1903
26.5000		0.0034	0.2611	22.1711
26.7500		0.0032	0.2432	22.1532
27.0000		0.0030	0.2266	22.1366
27.2500		0.0028	0.2110	22.1210
27.5000		0.0026	0.1965	22.1065
27.7500		0.0024	0.1830	22.0930
28.0000		0.0022	0.1704	22.0804
28.2500		0.0021	0.1587	22.0687
28.5000		0.0019	0.1477	22.0577
28.7500		0.0018	0.1375	22.0475
29.0000		0.0017	0.1280	22.0380
29.2500		0.0016	0.1192	22.0292
29.5000		0.0014	0.1109	22.0209
29.7500		0.0013	0.1032	22.0132
30.0000		0.0013	0.0960	22.0060
30.2500		0.0012	0.0894	21.9994
30.5000		0.0011	0.0832	21.9932
30.7500		0.0010	0.0774	21.9874
31.0000		0.0009	0.0720	21.9820
31.2500		0.0009	0.0670	21.9770
31.5000		0.0008	0.0623	21.9723
31.7500		0.0008	0.0579	21.9679
32.0000		0.0007	0.0539	21.9639
32.2500		0.0007	0.0501	21.9601
32.5000		0.0006	0.0466	21.9566
32.7500		0.0006	0.0434	21.9534
33.0000		0.0005	0.0403	21.9503
33.2500		0.0005	0.0375	21.9475
33.5000		0.0005	0.0349	21.9449
33.7500		0.0004	0.0324	21.9424

4.7.4 Comparison with the method proposed by CWC

CWC has derived a set of equations for UH parameters for this region for larger catchements (CWC,1991) and these equations (Eqns. 2.17 to 2.24) have been given in Subsec 2.5.1. The values of peak discharge/unit area (q_p) , time to peak (t_p) and peak discharge (Q_p) computed using the equations are :

q_p = 0.7664 m^3/sec/Km^2 ; t_p = 2.779 hours; and Q_p = 16.79 m^3/sec.

The values of Q_p obtained by the CWC approach is much higher than those obtained by the approach suggested in the present study or the Snyder's approach.

The set of equations derived by CWC is essentially based on data available for larger catchments and these do not seem to apply to the smaller catchments. The value of LL_c/S for this basin is 14.25. From the curve developed by CWC between LL_c/S and q_p, to get a value of q_p of the order of 0.7 m^3/sec/Km^2, the value of LL_c/S has to be of the order of 5000 which means that the slope S has to be very small, which is not realistic. The approach proposed by CWC, thus, seems to be valid only for larger basins and not smaller catchments.

C H A P T E R   5

USGS DISTRIBUTED ROUTING RAINFALL-RUNOFF MODEL

5.1 Introduction

The distributed parameter model can generally take into account the spatial and the topographic characteristics of the watershed. Hence it is a more realistic representation of the watershed. There are a large number of distributed parameter models for the watersheds (Fleming,1975, Beven,1985). A simple model to simulate runoff hydrographs was developed by the Water Resources Division of the United States Geological Survey (Dawdy et al ,1978). The model known as the Distributed Routing Rainfall Runoff Model (DR3M) is deterministic and it combines the soil moisture accounting and rainfall excess components developed by Leclerc and Schaake (1973). Wherever possible, a physical interpretation is placed upon the parameters used in the model. Though developed essentially for routing urban flood discharges through a branched system of pipes to natural channels, Murthy (1988) used this model for simulating flood hydrographs from a semiurban and rural catchment and a hilly catchment and concluded that this model is reasonable and realistic for such watersheds. Furthermore, it is a relatively simple model with physically based parameters. Hence it is proposed to use the USGS model in the study and the following description generally follows Dawdy et al (1978).

Inputs to the model include daily rainfall and rainfall during unit rainfall days on which major storm precipitation occurs, the duration being a multiple of 5 minutes from atmost three raingauge stations; daily pan evaporation, and a physical definition of the basin discretised into as many as 50 segments including overland flow, channel and reservoir segments. The model maintains a unit duration soil moisture accounting during rainy days and daily accounting on unit rainfall days.

5.2 Structure of the Model

The model is divided into four major components: a soil moisture or water balance component; rainfall excess component; a routing component and an optimising component. The overall flow chart of the model is given in Fig. 5.1.

5.2.1. Soil moisture accounting component

The schematic diagram of the soil moisture accounting component is shown in Fig.5.2. This component determines the effect of antecedent conditions on infiltration. Soil moisture is modelled as a two layer system, one representing the base moisture storage (BMS) and the other upper wetted part caused by infiltration into soil moisture storage (SMS).

During unit rainfall days, moisture is added to SMS based on Philip's infiltration equation (Philip, 1954)

$$\frac{di}{dt} = k\left(1 + \frac{P(\bar{m} - m_o)}{i}\right) \tag{5.1}$$

where i = accumulated infiltration volume in wetted soil column since the start of the infiltration;

k = capillary conductivity of soil;

P = capillary potential at wetting front in soil column at the time at which the infiltration rate is being computed;

m_o = uniformly distributed moisture content of soil column when infiltration started; and

m = moisture content, uniformly distributed through wetted column at the time at which the infiltration rate is being computed.

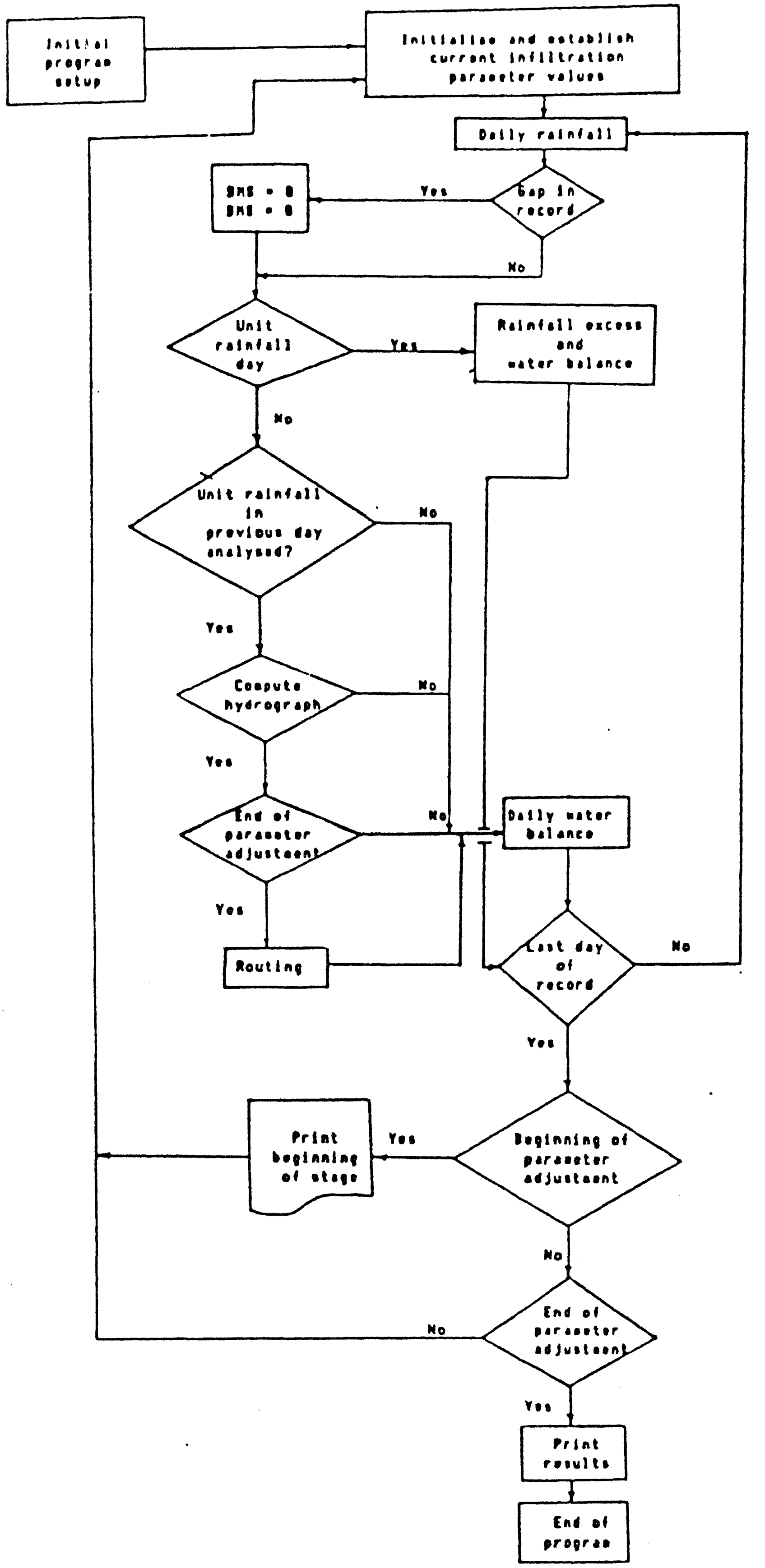


Fig. 5.1 Flow chart for DR3M (Adapted from Dawdy et al.,1978)

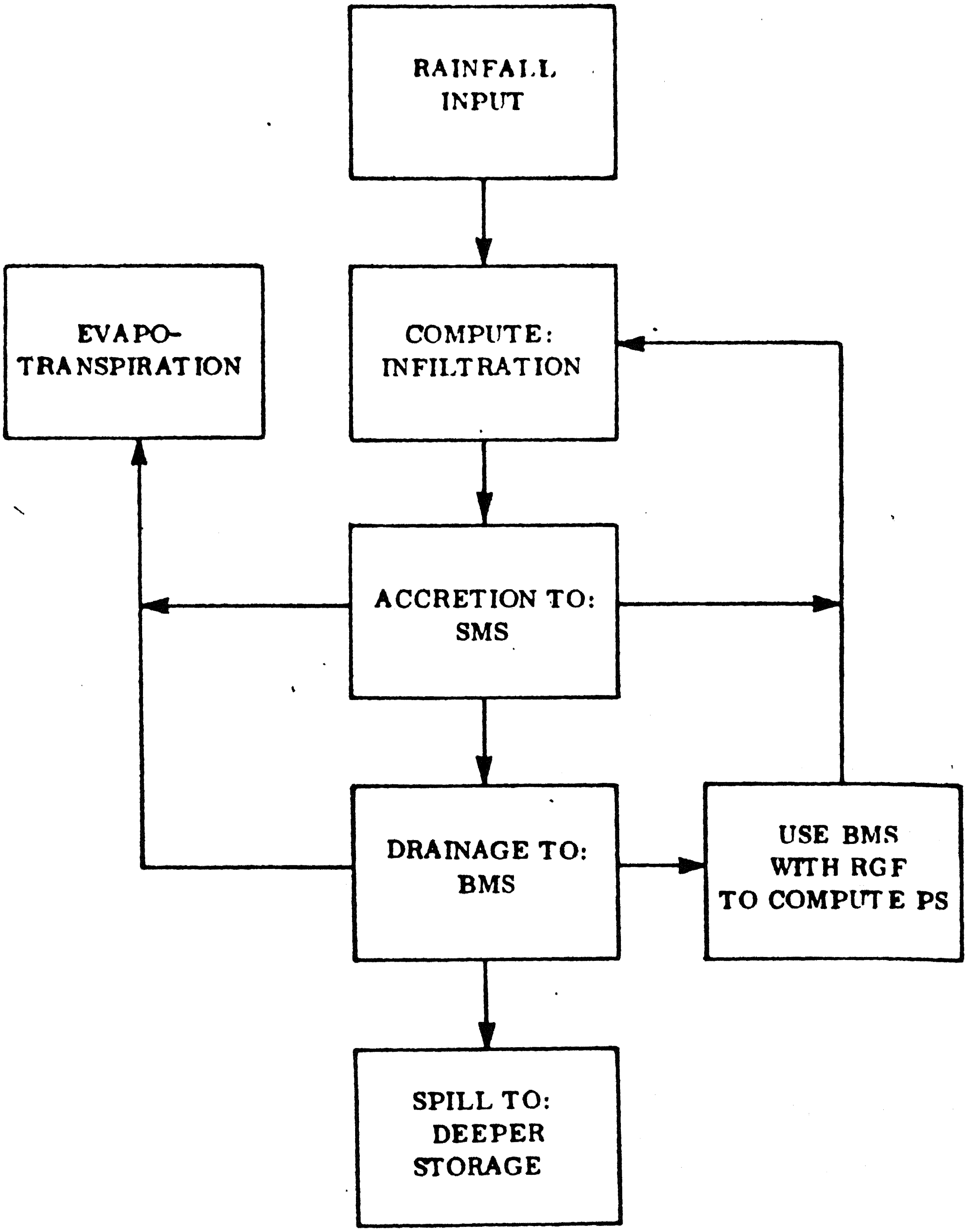


Fig. 5.2 Schematic flow chart of soil-moisture accounting component

It is assumed that at the beginning of infiltration, the entire soil column is at a uniform moisture content m_o and at any time during infiltration, the moisture content m above the wetting front is also uniform. Eventhough these assumptions do not give exact depth of the wetting front from the surface, they appear to give useful results (Philip, 1954).

On other days, a specified proportion of daily rainfall (RR) infiltrates into the soil. If the daily precipitation is less than the daily infiltration rate, the infiltration rate is reset equal to the the daily precipitation rate. Irrigation can be accounted for in the daily water balance as an additional input through supplied irrigation rates for each month.

Evapotranspiration takes place from SMS based on availability and otherwise from BMS with the rate determined from pan evaporation multiplied by a pan coefficient(EVC). Moisture in SMS drains into BMS with a controlling parameter (DRN) determining the rate. Storage in BMS has a maximum value (BMSN) equal to the field capacity moisture storage of the active soil zone. Zero storage in BMS is assumed to correspond to wilting point condition in the active soil zone. When storage in BMS exceeds BMSN, the excess is spilled to a deep storage. These spills could be the basis for routing interflow and baseflow components, if desired. However, this option is not included in the present model. Table 5.1 shows the various parameters for soil moisture accounting. Since the original data were generally in FPS units and the program was also in FPS units, it was decided not to modify the units in the program.

Table 5.1 Parameters for Soil-Moisture Accounting and Infiltration

Soil-Moisture Accounting

Parameters

DRN	=	A constant drainage rate for redistribution of soil moisture between SMS and BMS, in inches per day
EVC	=	A pan coefficient for converting measured pan evaporation to potential evapotranspiration
RR	=	The proportion of daily rainfall that infiltrates into the soil for the period of simulation excluding unit-rainfall days
BMSN	=	Soil-moisture storage at field capacity, in inches.

Infiltration

Parameters

KSAT	=	The effective saturated value of hydraulic conductivity, in inches per hour
RGF	=	Ratio of suction at the wetting front for soil moisture at wilting point to that at field capacity
PSP	=	Suction at wetted front for soil moisture at field capacity, in inches of pressure

5.2.2 Rainfall excess component

i) **Impervious surface:** Two types of impervious surfaces are considered by the model, namely, effective impervious surfaces and noneffective impervious surfaces. The effective impervious surfaces are those which are directly connected to the channel drainage system such as streets and roofs which drain onto driveways and streets. The noneffective impervious surfaces are those which drain onto the pervious areas.

The only abstraction from rainfall on effective impervious area is impervious retention, which is to be specified and must be filled before runoff from effective impervious areas can occur. Evaporation occurs from impervious retentions during periods of no rainfall.

Rain falling on non-effective impervious areas is assumed to run off to the surrounding pervious area. The model assumes that this occurs instantaneously and that the volume of runoff is uniformly distributed over the contributing pervious area. This volume expressed as inches over the pervious area is added to the rain falling on the pervious area prior to the computation of pervious area rainfall excess.

ii) Pervious surfaces: The point potential infiltration (FR) is computed by Philip's equation (Philip,1954) given by

$$FR = KSAT\ (1 + \frac{PS}{BMS}) \tag{5.2}$$

where KSAT is the effective saturated soil capillary conductivity and PS is the capillary potential at the wetting front. PS varies over the range from the field capacity to wilting point linearly as (Fig. 5.3)

$$PS = PSP\ (RGF - \frac{(RGF - 1)BMS}{BMSN}) \tag{5.3}$$

where PSP is the suction at wetting front at field capacity, RGF is ratio of suction at wilting point to that at field capacity and BMSN is the effective soil moisture storage at field capacity.

Point potential infiltration (FR) is converted to effective infiltration over the basin using a scheme for area variability of infiltration, first presented by Crawford and Linsley (1966). If SR represents the supply rate of rainfall for infiltration and QR represents the rate of generation of rainfall excess, the area

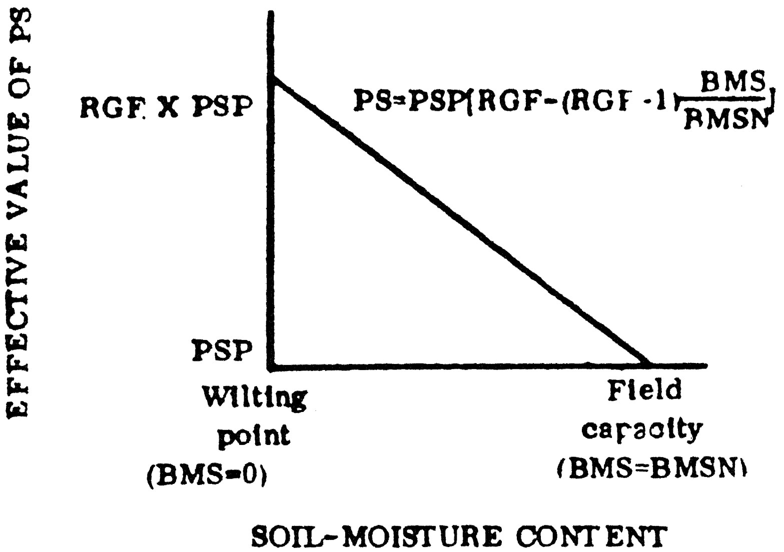


Fig. 5.3 Variation of soil-moisture potential

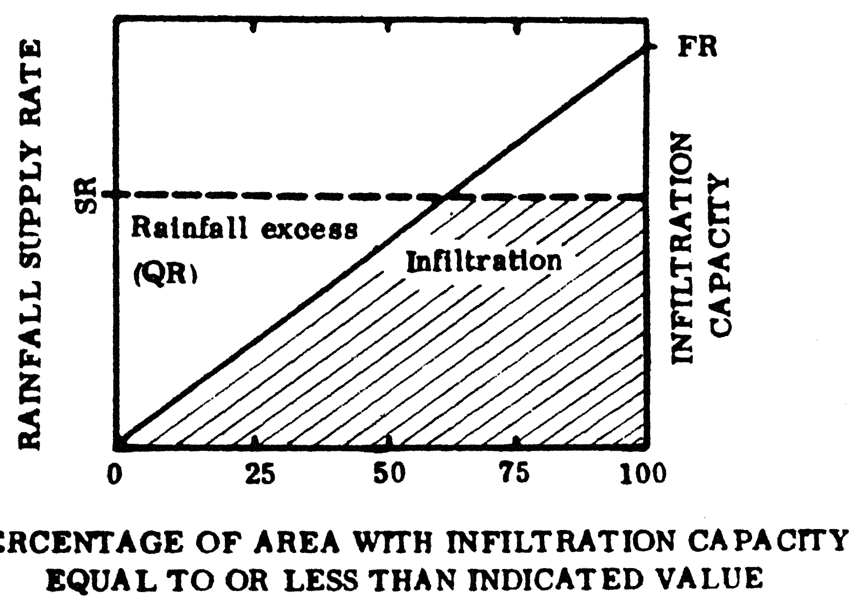


Fig. 5.4 Area variability model for infiltration

variability equations are

$$QR = SR^2/2FR \qquad SR < FR \tag{5.4 a.}$$

$$QR = SR - (FR/2) \qquad SR > FR \tag{5.4 b.}$$

Fig. 5.4 shows the schematic representation of these relationships. The infiltration parameters, described above, have been summarised in Table 5.1. Two different soil types can be handled by the model with separate soil moisture accounting and infiltration parameters for each soil type.

5.2.3 Routing component

A drainage basin is represented in this model as a set of segments which jointly describe all sub basins in the total basin. There are four basic types of segments: (i) overland flow segment, (ii) channel segment, (iii) reservoir segment, and (iv) nodal segment. The basin is divided into a number of segments in such a way that the essential basin geometry and physiography are taken into account for runoff computation.

(i) **Overland flow and channel segments:** A channel segment is permitted to receive upstream inflow from as many as three other segments, including combination of other channel segments, reservoir segments and nodal segments. It may also receive lateral inflow from as many as four overland segments. The overland flow segments receive uniformly distributed lateral inflow from excess precipitation.

Kinematic wave theory is used for both overland flow and channel routing. The partial differential equation to be solved for each channel and overland

segment is

$$\frac{\delta A}{\delta t} + \frac{\delta Q}{\delta x} = q \tag{5.5}$$

where A is the area of flow, Q is the rate of flow, q is the rate of lateral flow, t is time and x denotes the distance along the segment increasing in the downstream direction. The dependent variables A and Q are functions of two independent variables x and t. The relationship between A and Q is given by

$$Q = \alpha\ A^m \tag{5.6}$$

The length of segment is L and the outflow hydrograph from the segment is Q(L,T). The inflow hydrograph to the upstream end of the segment is Q(0,t) and these are the boundary conditions needed to solve Eqn.5.5. In case there is an upstream inflow, the solution will also depend on that inflow. Therefore, the solution must be some function of x, t, q and Q(0,t). The outflow hydrograph is given by

$$Q\ (L,t) = f\ (\ Q\ (0,t),\ q(t)\ ) \tag{5.7}$$

The two parameters α and m of the kinematic wave model can be determined by analysis of the physical characteristics of the basin segments. For example, if Chezy's equation is employed to quantify friction, then

$$\alpha = C\ (S_0)^{0.5} \text{ and } m = 3/2 \tag{5.8 a}$$

where C is the Chezy's friction factor and S_0 is the slope. If Manning's relationship is used,

$$\alpha = \frac{1}{n}\ (S_o)^{0.5} \tag{5.8 b}$$

where n is the Manning's friction factor. Functional relationships of α and m for channel and overland flow segments are listed in Table 5.2.

Numerical techniques are used to approximate Q(x,t) at discrete locations in the (x,t) plane. In this case, the rectangular grid of points was selected at intervals

Table 5.2 Relationships for estimating kinematic wave parameters α and m

Type of segment	ITYPE(I)	α	m	Definition of PARAM(I,
Rectangular conduit	1	$\frac{1.49\sqrt{SLOPE(I)}}{FRN(I)\ (PARAM(I,1)^{2/3})}$	1.67	width of conduit
Circular pipe	2	$\frac{1.49}{FRN(I)}\ (PARAM(I,1)/4)^{2/3}$ SLOPE(I)	1.0	diameter of pipe
Triangular cross-section	3	$\frac{1.49\sqrt{SLOPE(I)}}{FRN(I)\ (PARAM(I,1)^{1/3})}$	1.33	width at 1-foot depth
External specification of and m	4	PARAM(I,1)	PARAM(I,2)	
Overland flow (turbulent)	5 or 15	$\frac{1.49\sqrt{SLOPE(I)}}{FRN(I)}$	1.67	
Overland flow (laminar)	6 or 16	$\frac{66.4*\sqrt{SLOPE(I)}}{0.0000141*FRN(I)}$	3.0	

FRN(I) = Friction coefficient for segment I

SLOPE(I) = Slope of segment I

of time Δt and distance Δx. The value of Δx varies from segment to segment but the value of Δt is constant for all segments. Four points of a finite difference mesh are shown in Fig.5.5. The purpose of the finite difference scheme is to solve for A and Q at point d, given the values of Q and A at points a, b, and c.

The model contains two different finite difference equations and selects the appropriate one at each point in the solution. This is done in an attempt to keep the solution errors small while maintaining an unconditionally stable solution.

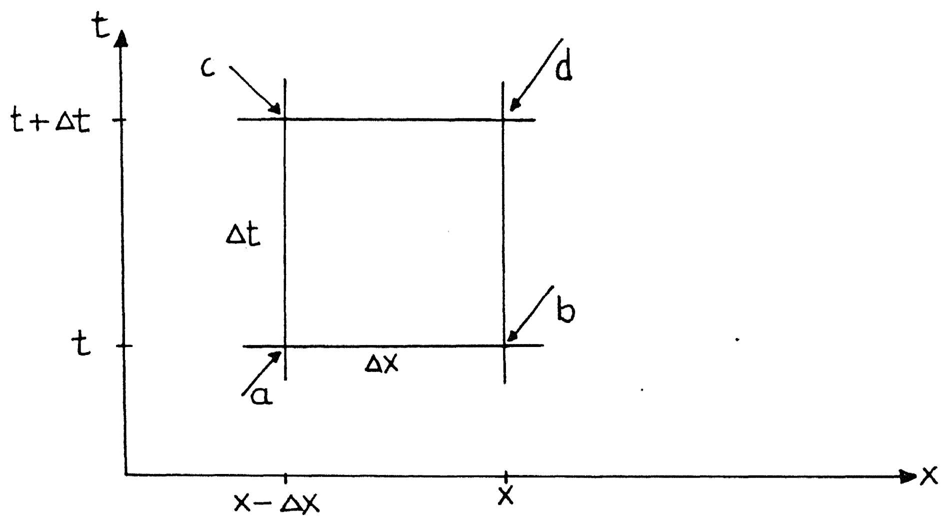


Fig. 5.5 Four point finite - difference mesh

The decision depends upon the parameter

$$\theta = m \frac{\Delta t}{\Delta x} \frac{Q_b}{A_b} = \alpha \ m \ A^{m-1} \left(\frac{\Delta t}{\Delta x}\right) \tag{5.9}$$

If $\theta > 1$, the equations used are

$$Q_d = Q_c + q \ \Delta x - \Delta x \ (A_c - A_a)/\Delta t \text{ , and} \tag{5.10}$$

$$A_d = (Q_d/\alpha)^{1/m} \tag{5.11}$$

This involves only mesh points a, c and d. It was derived by substituting $(A_c-A_a)/\Delta t$

for $\delta A/\delta t$ and $(Q_d-Q_c)/\Delta x$ for $\delta Q/\delta x$.
If $\theta < 1$, the equations used are

$$A_d = A_b + q\Delta t + \Delta t\ (Q_a + Q_b)\text{ , and} \tag{5.12}$$

$$Q_d = \alpha\ A_d^{\ m} \tag{5.13}$$

Initial values of A and Q are given along the entire X-axis. At t = 0, the model sets A = 0 and Q = 0 everywhere. During solution, the upstream inflows are given and Eqn 5.6 is used to compute the upstream boundary condition for A. Equations 5.10 to 5.13 are solved by the model beginning with x = 0 and proceeding downstream at intervals of Δx to x = L.

5.2.4 Selection of Δx and Δt

Dawdy et al (1978) use the following approach to select Δx and Δt. Two factors are important for Δt. One is the frequency of rainfall input to the catchment and the other is the frequency reponse characteristics of the catchment. For a basin having overland flow length L_o, overland kinematic wave parameter α_o and m_o, channel length L_c and channel flow parameters α_c and m_c,

$$\Delta t \simeq 0.1\ (t_o + t_c) \tag{5.14}$$

where

$$t_o = \left[\frac{L_o}{\alpha_o\ (i_e\ /\ 43200)^{m_o-1}} \right]^{1/m_o} \text{ seconds} \tag{5.15}$$

and
$$t_c = \left[\frac{L_c}{\alpha_c\ (\ N\ i_e\ L_o/43200)^{m_c-1}} \right]^{1/m_o} \text{ seconds} \tag{5.16}$$

where i_e is the maximum rainfall intensity in inches/hour, N is the number of sides of channel with overland flow segments contributing lateral inflow (1 or 2) and L_c and L_o are in feet.

After Δt is selected, Δx is chosen for each segment. This is done by specifying the number NDX of Δx segments in each segment. NDX may vary from segment to segment.

The finite difference solution will be an exact solution if Δx and Δt are selected so that characteristics passing through point a (Fig.5.5) also passes through point d. In such a case, solution A_d and Q_d depends only on A_a and Q_a and lateral inflow along the characteristics curve between a and d. In the special linear case when m = 1, an exact solution may be obtained by selecting the ratio $\frac{\Delta x}{\Delta t} = \alpha^* = \alpha m A^{m-1}$. Hence $\Delta x \simeq \alpha^* \Delta t$. α^* is the effective value of α that would be used for a linear approximation to a given kinematic flow segment. For overland flow, $\alpha_o^* = L_o/t_o$; For channel flow, $\alpha_c^* = L_c/t_c$. Thus, $\Delta x_o \simeq L_o \; \Delta t/\Delta x$; $\Delta x_c \simeq L_c \; \Delta t/\Delta x$; and $NDX_o = \frac{L_o}{\Delta x_o} \simeq \frac{t_o}{\Delta t}$ and $NDX_c = \frac{L_c}{\Delta x_c} \simeq \frac{t_c}{\Delta t}$.
In general, the smallest subarea of interest and the highest intensity of rainfall should be used to estimate Δt.

5.2.5 Optimisation component

An option is included in the model to calibrate the soil moisture and infiltration parameters for drainage basins having rainfall runoff data. The method of determining optimum parameter values is based on the technique devised by Rosenbrock (1960). The objective function is the sum of the squared deviations of the logarithms of computed and measured storm runoff volumes. To start the fitting process, the model is assigned an initial set of parameter values and upper and lower bounds of each parameter. The suggested values by Dawdy at al are shown in Table 5.3. The estimation of all the parameters simultaneously is complicated and unnecessary. The procedure involves estimation of parameters in a number of phases. Initially some of the less sensitive parameters are assumed constant and the more sensitive parameters are optimised. The process itself may be iterated in a number of phases. For example, the last three parameters (Table 5.3) were found to be less sensitive and assuming them as constant, the remaining parameters are optimised. While optimising the four parameters, each of them can be

Table 5.3 Initial values of soil-moisture and infiltration parameters and their bounds

Parameter	Initial value	Lower limit	Upper limit	Units
PSP	5.00	1.00	15.00	Inches
KSAT	0.10	0.01	1.00	Inches/hour
RGF	10.00	5.00	20.00	Dimensionless
BMSN	5.00	1.00	15.00	Inches
EVC	0.70	0.50	1.00	Dimensionless
RR	0.90	0.65	1.00	Dimensionless
DRN	0.50	0.10	1.00	Dimensionless

optimised separately keeping the rest constant till an acceptable result is obtained. The optimisation procedure revises the parameter magnitudes and recomputes the objective function using the revised set of parameter magnitudes. If the result is an improvement, the revised set is accepted; if not, the method returns to the previous best set of parameters.

It is found during the studies that the results depend very much upon the initial values of the parameter set and hence, the initial values should be chosen with care or alternate sets of initial values may be tried.

5.3 Implementation of the Computer Program

The original program developed by USGS Water Resources Division is written in FORTRAN IV and is available from the User's Manual for the model (Dawdy et al,1978). This program was implemented in the microVAX II computer system of the Centre for Water Resources Engineering and Management at IIT Kanpur by Murthy

(1988) and was available for the study. Some minor modifications were to be made while implementing the program, since the original program was developed for CDC 7600 machine.

5.4 Analysis Using DR3M

DR3M requires the basin to be divided into a consistent set of overland flow and channel segments. A main channel runs through the entire length of the basin with its tributaries. For convenience, the main channel is divided into a certain number of segments and the tributaries are treated as separate channel segments. While delineating these segments the number of segments is to be balanced; uniformity within segments is to be maintained as far as possible; segmentation of overland flow is to be done with balanced length of overland flow to the designated channel segments; and compatibility between overland flow segments and the designated channel segments is also to be maintained. The area of each segment is planimetered and slopes are determined from contours in a topo sheet of the basin. The length of overland flow is computed by dividing the area of each segment by the length of the channel segment into which it contributes as a lateral flow. Crosssections of the river channel in different reaches are required for the analysis. The crosssection data are generally available only at the bridge sites for the basins selected for the study. Two options are available for the roughness coefficient for the overland flow segments. A single roughness coefficient can be assigned to an overland flow segment or else the segment can be assigned two roughness coefficients, one for the pervious and the other for the impervious surfaces.

Direct impervious areas in the basin are to be estimated with care because

they contribute directly to the channel without any loss (Rao and Ramaseshan, 1978). In the absence of data, particularly of rocky and hilly impervious areas in these basins, the impervious area is initially assumed to be negligible and ignored. An attempt is made to parameterise this factor in the final stages of the study. In case data were available, it will be desirable to identify the spatial distribution of such areas and use them appropriately in the model.

5.4.1 Analysis of a basin using DR3M

To illustrate the details of the analysis of a basin using DR3M, the details of the Talma river basin is presented. This basin (Fig.5.6) has a catchment area of 42.12 Km^2 and has one raingauge station. The data for this basin are available generally from 15 May to 15 October for the years 1965 to 1969. Since evaporation data were not available for any station within the basin, pan evaporation was calculated for Guwahati which is the nearest station for which data were available and considered reasonably representative of the basin. Christiansen's equation was used in the estimation of monthly pan evaporation with monthly normal values of the meteorological characteristics of Guwahati. A pan coefficient of 0.7 was used to estimate the monthly potential evaporation. Daily normal potential evaporation was used in the model.

The basin is divided into a limited number of overland flow and channel segments (Fig.5.6). A main channel runs through the entire basin and there is a major tributary. The main channel is divided into two channel segments CH20 and CH21 respectively and the major tributary is designated as channel segment CH22. On either side of the channel segments, there are overland flow segments and they are designated as OF01 to OF06. Direct lateral inflow into downstream channel

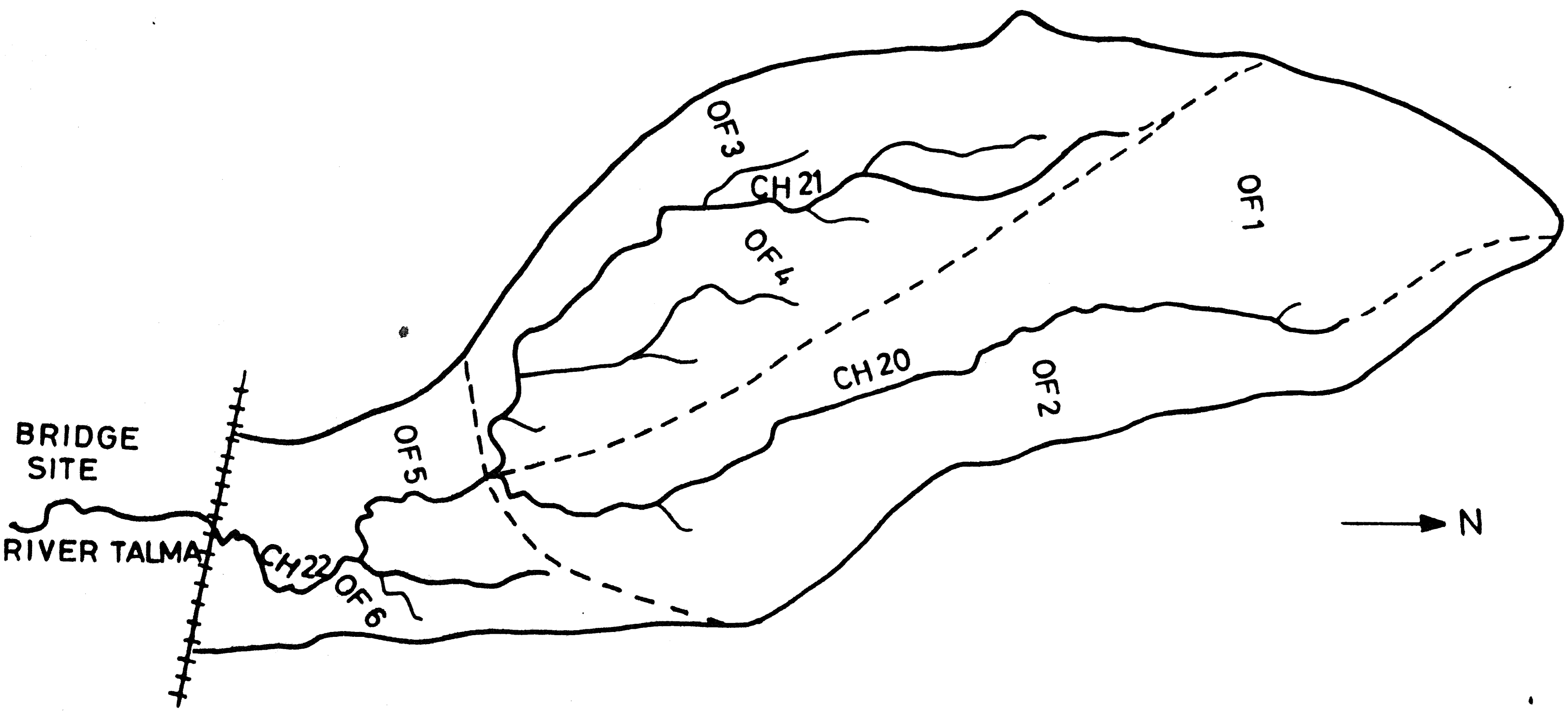


FIG. 5.6 SEGMENTATION OF TALMA BASIN

section is ignored. Thus the entire basin is divided into 9 segments consisting of 3 channel segments and 6 overland flow segments. The details of these segments, for example, the length and slope of the channel segments and the length, area and average slope of the overland flow segments are determined from topo sheet. The length of overland flow was computed by dividing the area by the length of the channel segment into which it contributes as lateral inflow.

The crosssection of the river channel in different reaches are required. However generally details of only one crosssection, viz., that at gauging station was available. While it may be possible to measure the crosssectional details at other upstream sites, it was not possible to do so in the study because of limitations of time, resources and remoteness of the region. From a comparison of a large number of crosssections of small rivers in the regions, approximate cross- sections for upstream channel segments were adopted in the study. The channel and overland flow segment characteristics of the basin are shown in Table 5.4.

Since rainfall and runoff data are available from 15 May, and May is a hot month with very limited rainfall, the simulation is started every year from the beginning date of record with the initial condition of the basin corresponding to a very dry condition. A warming up period of about 15 days in the month of May brings the soil moisture, baseflow etc., to the realistic conditions and the model fits only important floods generally between July and September. In the case of Talma basin, initially 12 storm flood events were identified for use in the study (Sec.3.5).

Table 5.4 Segment characteristics of Talma basin

Channel characteristics

Segment No.	Upstream segments	Length feet	Slope ft/ft	Width ft	Manning's n
CH20		29040	0.007	80	0.035
CH21		27456	0.007	90	0.035
CH22	CH20,CH21	12672	0.009	110	0.040

Overland flow characteristics

Segment No.	Channel segment for drainage	Length feet	Slope ft/ft
OF01	CH20	4439	.0047
OF02	CH20	3351	.0063
OF03	CH21	3223	.0047
OF04	CH21	2486	.0060
OF05	CH22	2127	.0056
OF06	CH22	3291	.0037

It should be noted that in this model, the simulation is done not for a storm event but for all the storms in a year sampled continuously over time. The initial simulation used the parameters and bounds (Table 5.3) suggested by Dawdy et al, viz., PSP=10; KSAT = 0.10; RGF=10; BMSN= 5.0; EVC = 0.70; RR=0.90; and DRN = 0.50. The results of the analysis are shown in columns 5 and 6 in Table 5.5. A comparison of observed and simulated floods indicate that the simulated volume of runoff are generally much higher and the simulated peaks are also higher except for small flood events 2, 3, and 5. The time to peak of the simulated flood events is comparable to that of observed data except for storm events 3, 4, 7, 9, 11 and 12.

TABLE 5.5 RESULTS OF ANALYSIS FOR TALMA BASIN

FLOOD NUMBER	OBSERVED VALUES			NO OPTIMISATION			SIMULATED VALUES: OPTIMISATION WITH INITIAL VALUES		SIMULATED VALUES: OPT. WITH PSP=10		SIMULATED VALUES: OPT. WITH PSP=10 RGF=18		SIMULATED VALUES: OPT. WITH PSP=10 RGF=18;BMSN=10		SIMULATED VALUES: OPT. WITH PSP=10 RGF=18;BMSN=10	
							ALL FLOODS		ALL FLOODS		ALL FLOODS		ALL FLOODS		FLOODS 4,7,9,10,11	
	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	179	0.147	6	379	0.645	6	161	0.330	135	0.286	135	0.287	134	0.285	119	0.260
2	396	0.732	13	330	0.627	12	113	0.256	100	0.233	100	0.233	99	0.231	80	0.195
3	739	1.548	13	598	1.166	9	261	0.550	229	0.491	225	0.482	225	0.982	200	0.436
4	4704	8.401	14	6180	10.775	12	5295	8.939	5184	8.738	5190	8.750	5178	8.726	4979	8.324
5	251	0.418	16	219	0.422	17	76	0.182	66	0.163	62	0.157	63	0.158	55	0.143
6	501	0.710	11	1316	2.560	10	896	1.678	857	1.609	860	1.613	852	1.599	702	1.335
7	1213	2.359	16	1912	4.228	11	1342	2.892	1308	2.824	1311	2.829	1301	2.810	1125	2.463
8	625	0.357	7	1138	1.663	7	539	0.876	510	0.838	469	0.784	477	0.795	430	0.728
9	3093	5.997	22	4791	7.190	7	4262	6.241	4217	6.166	4221	6.173	4207	6.149	3889	5.556
10	3523	9.741	35	4299	12.323	32	3197	8.151	3148	8.051	3154	8.073	3131	7.989	2829	6.965
11	2703	3.126	11	3926	6.783	7	3133	5.271	2982	5.031	2984	5.034	2977	5.021	2861	4.799
12	598	0.65	14	1190	2.034	8	638	1.165	643	1.166	646	1.170	636	1.560	511	0.958
				PSP				8.179		10.000		10.000		10.000		10.000
				KSAT				0.167		0.163		0.163		0.164		0.182
				RGF				16.630		6.188		18.000		18.000		18.000
				BMSN				7.731		7.312		8.198		10.000		10.000
OBJECTIVE				INITIAL	8.699			8.699		7.447		7.534		7.442		0.343
FUNCTION VALUES				FINAL				5.762		5.999		6.011		6.009		0.303

TPEAK = TIME TO PEAK IN HOURS; PEAK = PEAK DISCHARGE IN CUSECS

EFF.RAIN = EFFECTIVE RAINFALL IN INCHES

Events 3 and 12 are small floods and flood event 9 has multiple peaks. However, time to peak is not a major parameter in this study.

There are seven major parameters in the model that can be fitted. Some of these can be treated as regional constants or may be evaluated separately from hydrograph analysis, seasonal water balance etc. Since the model is relatively insensitive to the three soil moisture accounting parameters, DRN, EVC, and RR, they are considered to be constants. Based on the soil characteristics, DRN, the constant drainage rate for redistribution of soil moisture between SMS and BMS of 0.5 inches/day; EVC, the pan coefficient for converting measured pan evaporation to potential evapotranspiration of 0.70; and RR, the proportion of rainfall that infiltrates into the soil for the period of simulation excluding unit rainfall days of 0.90, are adopted in the study. It is proposed to consider in greater detail the other four parameters in the modelling of the small watersheds.

Initially all the 12 storm flood events were considered to be equally important and the sum of the squared errors between the observed and simulated floods for all the flood events was considered to be a measure of goodness of fit of the model. Using the optimisation option, the four parameters were optimised from the initial values. A summary of the results of this analysis is shown in column 8 and 9 of Table 5.5. The results are poor for the relatively small floods 2, 3, and 5 as in the case of initial parameters and are otherwise much better for the larger floods. The values of the parameters and the initial and final values of the objective function are also shown in the Table. The objective function decreases slightly from around 8.7 to around 5.8.

A number of studies were carried out to understand the relative importance and sensitivity of each of the four model parameters. They included the

optimisation of various combinations of parameters and for different combination of floods, say all large floods and all small floods. The results indicate that the volume of runoff is significantly affected by the parameter KSAT and the results were generally less sensitive to PSP, RGF, and BMSN. Further studies are reported later.

5.4.2 Analysis of all the basins using DR3M

For all the basins used in the study, a few assumptions were made. Since no evaporation data were available, the information available at Guwahati was used (Sec. 5.4.1) and the monthly potential evaporation was estimated for each of the basins using the monthly normal values of the meteorological characteristics of Guwahati. Daily potential evaporation values were used in the model. The cross-sectional details were available only at the bridge site. Approximate crosssections for the various upstream segments were adopted in the study from a comparison of a large number of crosssections of small rivers in the region.

i) Batjhora Basin : This basin (Fig.5.7) has a catchment area of 7.33 Km^2 and the data were available for the years 1964 and 1965. The basin has one raingauge station. The main channel is divided into 4 channel segments , CH20, CH21, CH22, and CH23 and there are 8 overland flow segments, one either side of each of the channel segments and designated as OF01 to OF08.

There are 9 storm flood events in the basin. Assuming the initial values of the parameters as suggested by Dawdy et al, simulation was carried out and the results are shown in Table 5.6. A comparison of observed and simulated floods indicate that the simulated volumes of runoff are generally much higher and the

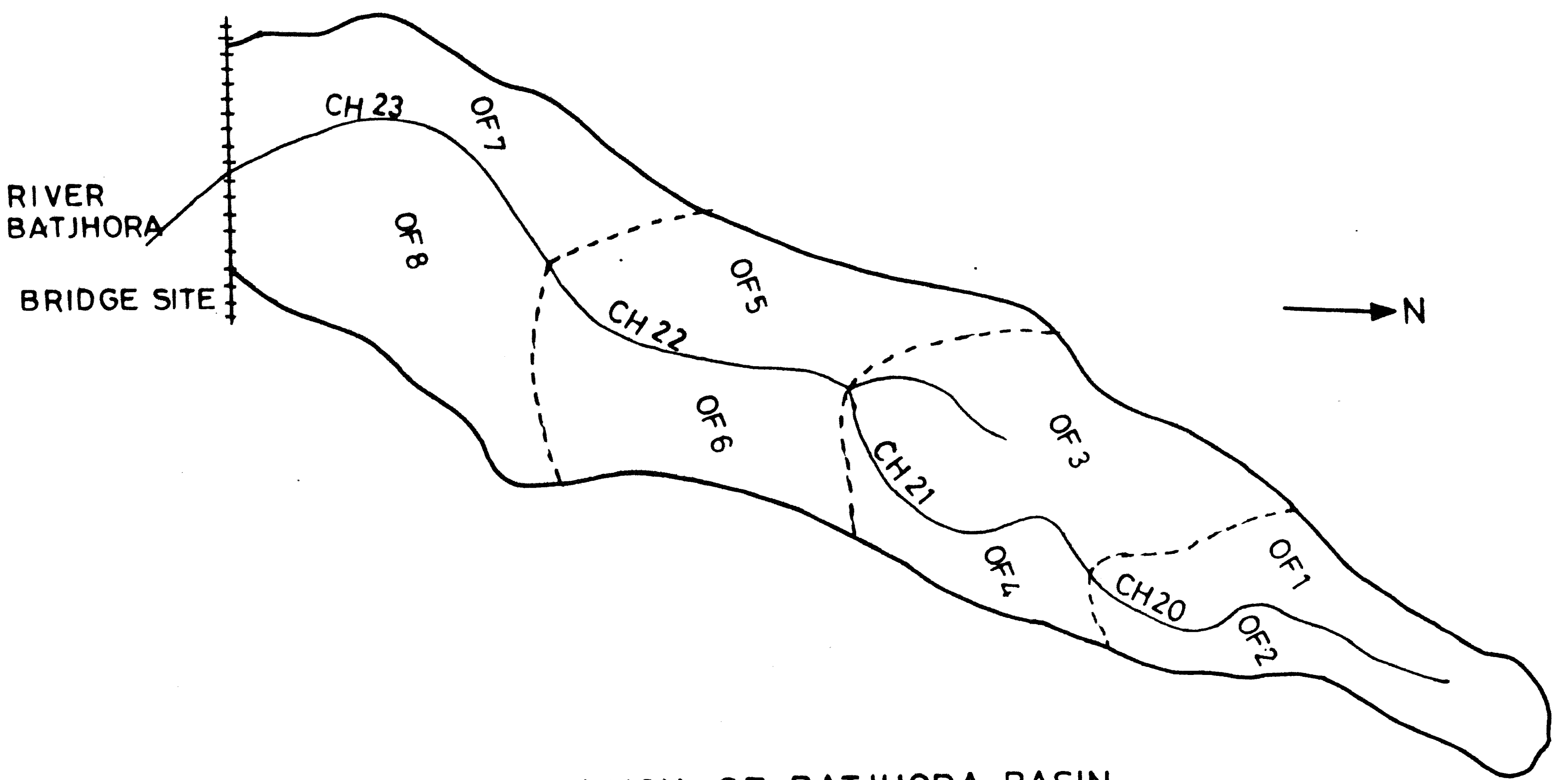


FIG. 5·7 SEGMENTATION OF BATJHORA BASIN

TABLE 5.6 RESULTS OF ANALYSIS FOR BATJHORA BASIN

LOOD MBER	OBSERVED VALUES		NO OPTIMISATION		SIMULATED VALUES: OPTIMISATION WITH INITIAL VALUES ALL FLOODS		OPT. WITH PSP=10 RGF = 18 ALL FLOODS		OPT. WITH PSP=10 ALL FLOODS		OPT. WITH PSP=10 RGF=18;BMSN=10 ALL FLOODS	
	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	160	0.356	365	0.500	223	0.315	229	0.321	270	0.37	229	0.321
2	399	1.322	2601	2.888	2443	2.574	2464	2.614	2543	2.673	2464	2.614
3	310	0.848	609	1.153	424	0.781	469	0.848	531	0.963	469	0.848
4	233	0.590	466	0.804	311	0.541	273	0.478	349	0.632	278	0.486
5	310	1.399	1390	1.324	1194	1.043	1116	0.955	1130	0.986	1116	0.956
6	813	3.779	1402	3.137	1286	2.550	1260	2.440	1288	2.568	1260	2.44
7	813	2.470	891	2.332	805	2.044	790	1.997	812	2.064	790	1.998
8	163	0.567	912	1.424	631	1.046	567	0.939	574	1.001	566	0.94
9	194	0.385	745	0.824	248	0.395	250	0.398	269	0.432	361	0.498
			PSP			7.586		10.000		10.000		10.000
			KSAT			0.112		0.102		0.096		0.102
			RGF			16.460		17.774		18.000		18.000
			BMSN			1.890		2.400		1.180		10.000
OBJECTIVE			INITIAL	2.380		2.380		1.222		0.300		1.226
FUNCTION VALUES			FINAL			1.126		1.157		0.220		1.215

TPEAK = TIME TO PEAK IN HOURS PEAK = PEAK DISCHARGE IN CUSECS

EFF.RAIN = EFFECTIVE RAINFALL IN INCHES

simulated peaks are also high for all the floods indicating that the system parameters are different from the initial values. Keeping the three soil moisture accounting parameters (EVC, RR, DRN) constant, the remaining four parameters, viz., the infiltration parameters PSP, KSAT and RGF and the soil moisture accounting parameter BMSN were optimised from the initial values. The results are shown in Table 5.6. The objective function value decreased from 2.380 to 1.126. It is found that except for flood events 2,5,6, and 8, the fit with optimal parameters is much better than that with initial parameters. The flood events 2, 5, 6 and 8 indicate very large differences that may be due to very high abstractions or data errors.

ii) **Sankhini Basin:** This basin (Fig.5.8) has a catchment area of 7.76 Km^2. The data for this basin are available only for the period 1964 and 1965. This basin has one raingauge station. The main channel is divided into 4 channel segments - CH20, CH21, CH24, and CH27. There are four tributaries to the main channel and these form 4 more channel segments, viz., CH22, CH23, CH25, and CH26. Thus there are 8 channel segments and correspondingly 16 overland flow segments, OF01 to OF16.

There are 5 storm flood events in the basin. The results of the analysis are shown in Table 5.7. The results of the simulation run assuming the initial parameters for the watershed show that both the simulated peak and volume of runoff are much higher than the observed ones. The results of the optimisation run are shown in columns 6 and 7 of the Table. The objective function has decreased significantly from 4.850 to 0.632 and there is a much better agreement between the peak values and the effective rainfall values for the observed and simulated floods. For floods 1 and 3, the simulated volume of runoff are higher while for the other 3

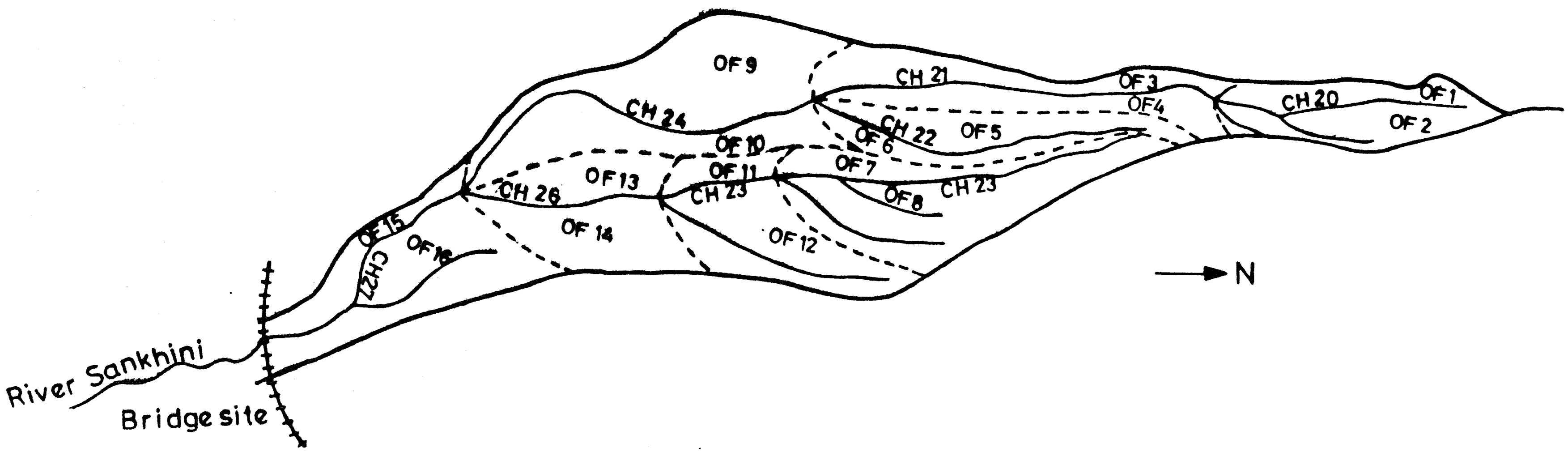


FIG. 5·8 SEGMENTATION OF SANKHINI BASIN

TABLE 5.7 RESULTS OF ANALYSIS FOR SANKHINI BASIN

		FLOOD NUMBER	OBSERVED VALUES PEAK	EFF.RAIN	NO OPTIMISATION PEAK	EFF.RAIN	OPTIMISATION WITH INITIAL VALUES ALL FLOODS PEAK	EFF.RAIN	OPT. WITH PSP=10 ALL FLOODS PEAK	EFF.RAIN
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
		1	338	1.666	734	4.526	376	2.187	475	2.789
		2	263	1.869	444	3.359	155	1.142	134	1.007
		3	279	2.529	676	7.855	450	3.776	481	3.990
		4	168	1.149	288	2.570	124	1.052	170	1.443
		5	132	0.642	336	2.251	64	0.437	54	0.373
	PSP							3.052		10.000
	KSAT							0.366		0.174
	RGF							17.404		6.290
	BMSN							12.591		8.020
OBJECTIVE	INITIAL							4.850		4.120
FUNCTION VALUES	FINAL							0.632		1.200

S I M U L A T E D   V A L U E S

		OPT. WITH PSP=10 RGF=18 ALL FLOODS PEAK	EFF.RAIN	OPT. WITH PSP=10 RGF=18 FLOODS 2,4 PEAK	EFF.RAIN	OPT. WITH PSP=10 RGF=18 FLOODS 1,3 PEAK	EFF.RAIN	OPT. WITH PSP=10 RGF=18;BMSN=10 ALL FLOODS PEAK	EFF.RAIN	OPT. WITH PSP=10 RGF=18;BMSN=10 FLOODS 2,4	
	PSP	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
	KSAT										
	RGF	485	2.848	641	3.827	268	1.576	402	2.346	651	3.894
	BMSN	136	1.023	230	1.680	88	0.675	114	0.867	249	1.821
OBJECTIVE	INITIAL	483	4.028	551	5.321	387	2.977	449	3.602	559	5.487
FUNCTION VALUES	FINAL	173	1.464	206	1.759	111	0.947	146	1.236	210	1.799
		51	0.351	63	0.431	31	0.221	43	0.297	65	0.44
			10.000		10.000		10.000		10.000		10.000
			0.171		0.139		0.277		0.208		0.135
			18.000		18.000		18.000		18.000		18.000
			8.740		10.395		8.045		10.000		10.000
			4.120		0.662		1.999		2.660		1.629
			1.290		0.192		0.296		0.837		0.201

TPEAK = TIME TO PEAK IN HOURS PEAK = PEAK DISCHARGE IN CUSECS
EFF.RAIN = EFFECTIVE RAINFALL IN INCHES

floods, they are lower. It may be seen that higher effective rainfall is associated with higher floods peaks (flood events 1 and 3) and lower effective rainfall is associated with lower flood peaks. Apparently, the infiltration conditions for these set of floods are different.

iii) Mujnai Basin: Mujnai river basin (Fig.5.9) has a catchment area of 38.40 Km^2. The data for this basin were available for the years 1964 and 1965 and this basin has a raingauge station. A main channel runs through the basin and it has one major tributary. The main channel is divided into 4 channel segments and the tributary forms the fifth channel segment. Thus this basin has 5 channel segments CH20, CH21, CH22, CH23, and CH24 and correspondingly 10 overland flow segments, OF01 to OF10.

This basin has 10 storm flood events during the period of record. The results of the analysis are shown in Table 5.8. The simulation run with the initial values of the parameters (columns 4 and 5) indicates that all but one of the simulated peaks and all the volumes of runoff are higher than the observed ones. The optimisation run with the initial parameters was carried out; the objective function decreases significantly from 12.081 to 1.563 and the results appear to be much better. Flood events 1,,3, and 9 show a higher simulated values; 4, 6, 7 and 8 have lower simulated values while flood events 2, 5 and 10 have comparable values. Thus as in the case of Sankhini river basin, this basin also seem to have at least two distinct set of flood events, one having a higher infiltration condition and the other lower infiltration condition.

iv) Khaga Basin : This basin (Fig.5.10) has a catchment area of

TABLE 5.8 RESULTS OF ANALYSIS FOR MUJNAI BASIN

SL.NO	OBSERVED VALUES		SIMULATED VALUES									
			NO OPTIMISATION		OPTIMISATION WITH INITIAL VALUES ALL FLOODS		OPT. WITH PSP=10 ALL FLOODS		OPT. WITH PSP=10 RGF=18 ALL FLOODS		OPT. WITH PSP=10 RGF=18;BMSN=10 FLOODS 5,6,7,8,10	
	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	1359	1.313	4704	3.722	3283	2.091	3478	2.278	4637	3.627	3731	2.531
2	851	0.932	3145	3.105	967	0.898	1265	1.098	3041	2.989	1463	1.263
3	1851	1.778	6977	5.227	5981	3.997	5970	3.983	6897	5.113	6167	4.196
4	1372	0.938	1612	1.049	641	0.519	634	0.514	1474	0.981	726	0.587
5	1048	0.775	1818	1.366	990	0.759	980	0.752	1767	1.332	1142	0.867
6	579	0.490	878	1.178	408	0.566	404	0.560	846	1.141	474	0.670
7	3028	2.181	2990	3.364	2220	2.393	2209	2.380	2962	3.326	2421	2.622
8	341	0.537	1901	2.052	314	0.310	302	0.327	1766	1.859	361	0.382
9	387	0.658	1925	2.172	696	0.62	683	0.609	1795	1.998	815	0.730
10	1062	1.078	2135	2.600	1149	1.088	1138	1.075	2044	2.447	1282	1.261

	PSP		9.909	10.000	10.000	10.000
	KSAT		0.198	0.199	0.103	0.173
	RGF		19.416	14.234	18.000	18.000
	BMSN		1.025	4.690	5.167	10.000
OBJECTIVE	INITIAL	12.081	12.081	9.364	9.364	1.450
FUNCTION VALUES	FINAL		1.563	1.620	8.493	0.505

PEAK = PEAK DISCHARGE IN CUSECS EFF.RAIN =EFFECTIVE RAINFALL IN INCHES

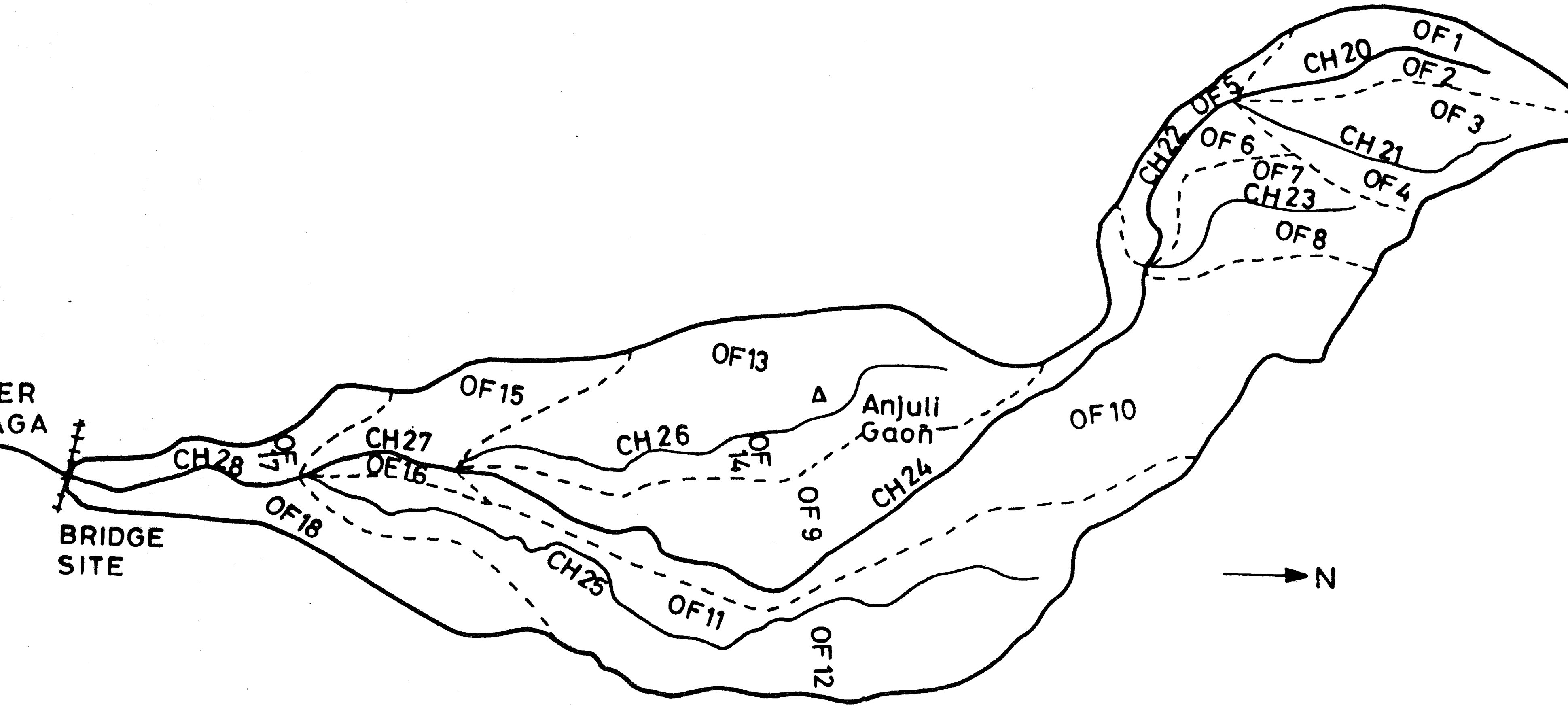


FIG. 5·10 SEGMENTATION OF KHAGA BASIN

66.00 Km^2 and has two raingauge stations. The data for this basin are available for the years 1981 and 1984 and from this, 5 flood events were identified for analysis. The main channel was divided into 5 channel segments and there are four tributaries which form another 4 channel segments. Thus there are 9 channel segments, CH20 to CH28 and 18 corresponding overland flow segments, OF01 to OF18. The results of the analysis are shown in Table 5.9.

The simulation run with the initial parameters showed higher values for flood peaks and volumes compared to the observed ones for flood events 2, 3, 4, and 5 while in the case of flood event 1, the simulated values were very much lower. Optimisation with the initial values of the parameters showed that for flood events 1 and 3, the simulated values were very much lower than the observed ones and for flood events 4 and 5, they were very much higher while for the flood event 2, the simulated and observed values were comparable (Table 5.9). The objective function value has decreased from 5.255 to 3.265 and yet is large. Further analysis with several alternate parameters indicated that in case flood event 1 were to be modelled well, there were very large differences in the case of all other floods. This indicates data errors in flood event 1. Simulation with a number of other parameter combinations indicated higher peaks. A comparison of times to peak indicated the need for introducing additional attenuation and time delay through storage elements in the river and this is considered in Subsec. 5.4.3.

v) Dharsi basin : This basin (Fig. 5.11) has a catchment area of 91.40 Km^2. The data for this basin are available for the years 1968 to 1970 and 12 storm flood events were identified. This basin has two raingauge stations. The main channel is divided into 5 channel segments, CH20, CH23, CH24, CH24, and CH27. There are 3 major

TABLE 5.9 RESULTS OF ANALYSIS FOR KHAGA BASIN

FLOOD NUMBER	OBSERVED VALUES			NO OPTIMISATION					
				K = 0			K = 2HOURS		
	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	TPEAK
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	13365	5.626	7	6863	3.443	5	9080	3.443	7
2	5371	2.106	12	13605	4.692	2	12859	4.692	11
3	1466	0.615	9	1960	0.957	6	2754	0.957	9
4	1999	1.369	13	6798	4.731	10	7501	4.731	12
5	1319	0.840	8	6854	4.262	6	7575	4.262	8

S I M U L A T E D V A L U E S

		OPTIMISATION WITH INITIAL VALUES ALL FLOODS		OPT. WITH PSP=10 ALL FLOODS		OPT. WITH PSP=10 RGF=18 ALL FLOODS		OPT. WITH PSP=10 RGF=18;BMSN=10 ALL FLOODS		OPT. WITH PSP=10 RGF=18 FLOODS 2,3,4,5		OPT. WITH PSP=10 RGF=18;BMSN=10 FLOODS 2,3	
		PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN				
		(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
		4906	2.198	4627	2.089	4724	2.134	5561	2.341	3615	1.632	10253	4.655
		5340	2.464	5119	2.376	5208	2.415	7564	3.475	3896	1.841	12241	5.643
		794	0.421	773	0.422	676	0.393	162	0.128	442	0.277	1085	0.563
		3914	2.894	4132	2.937	4166	2.959	4643	3.488	3559	2.349	8921	7.142
		4652	3.048	4924	3.375	4962	3.422	5190	3.724	4588	3.000	6241	5.411
	PSP		3.147		10.000		10.000		10.000		10.000		10.000
	KSAT		0.369		0.172		0.165		0.123		0.240		0.024
	RGF		18.286		6.250		18.000		18.000		18.000		18.000
	BMSN		6.310		6.874		6.000		10.000		6.087		10.000
JECTIVE	INITIAL		5.255		4.465		4.466		5.860		3.962		2.339
NCTION VALUES	FINAL		3.265		3.655		3.726		5.821		6.087		0.979

TPEAK = TIME TO PEAK IN HOURS PEAK = PEAK DISCHARGE IN CUSECS
EFF.RAIN = EFFECTIVE RAINFALL IN INCHES

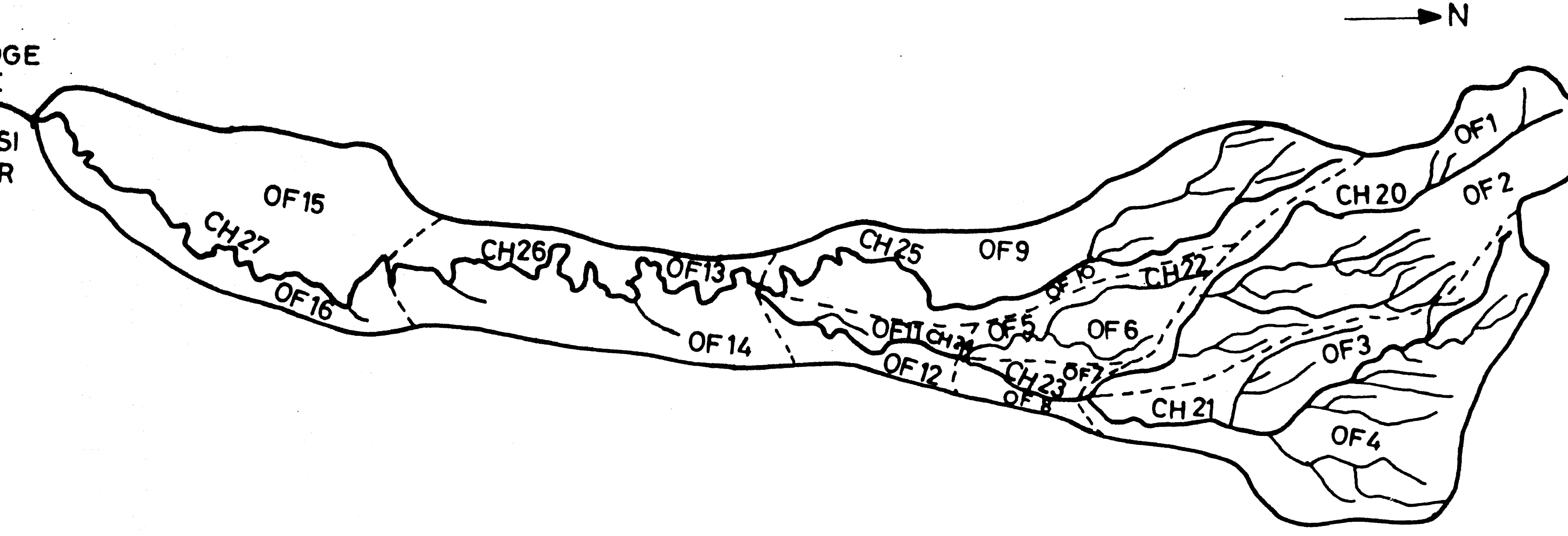


FIG. 5·11 SEGMENTATION OF DHARSI BASIN

tributaries joining the main channel at various locations and they are designated as channel segments CH21, CH22, and CH25. Thus there are 8 channel segments and correspondingly 16 overland flow segments, OF01 to OF16.

The results of analysis are shown in Table 5.10. The initial simulation run showed that all the simulated values were very much higher than the observed ones. A comparison of times to peak, as in the case of Khaga basin, indicated the need for introducing additional attenuation and time delay through storage elements in the river. The optimisation run (columns 11 and 12) reduced the objective function from 31.897 to 6.707. Also for flood events 1, 2, 6, and 11, the simulated values were lower while for the others, it was higher. Only for flood event 7, they are comparable. Thus for this basin also, two distinct groups of flood events with different infiltration conditions were identified.

vi) Mansari basin: This is the largest among the basins considered in th study. The catchment area of the basin (Fig. 5.12) is 213.00 Km^2 and it has raingauge stations. The data for this basin were available for the years 1970 an 1971 and 5 storm flood events were identified. The main channel is divided into channel segments, CH21, CH22, CH24, CH26, CH28, CH30. 5 major tributaries drai into the main channel at various locations and each of these form a channel segmer and they are designated CH21, CH23, CH25, CH27, and CH29. Thus there are 1 channel segments and 22 corresponding overland flow segments in this basin.

The results of the analysis are shown in Table 5.11. Simulation with initi values of parameters showed that for all the floods, the simulated values of tl peak were very much higher compared to the observed peaks. The volumes runoff were also high except for flood number 1. The simulation run wi

TABLE 5.10 RESULTS OF ANALYSIS FOR DHARSI BASIN

FLOOD NUMBER	OBSERVED VALUES			NO OPTIMISATION					
	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	TPEAK
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	743	0.499	13	5678	2.814	6	3421	2.814	12
2	3037	3.548	35	7456	7.414	7	3766	7.414	34
3	2522	3.082	39	10445	8.047	15	4270	8.047	27
4	1379	1.291	15	5879	7.601	6	3613	7.601	11
5	1245	0.781	13	10287	7.047	4	3841	7.047	21
6	1713	1.178	17	5645	2.653	13	3398	2.653	18
7	880	0.717	10	5056	1.941	5	2930	1.941	9
8	1773	1.059	14	9138	7.508	30	3810	7.508	33
9	1658	1.001	11	8375	4.378	4	3692	4.378	11
10	582	0.434	9	6492	4.449	5	3621	4.449	10
11	688	0.532	16	5505	2.357	8	3201	2.357	14
12	702	0.484	16	7617	3.717	9	3640	3.717	14
OBJECTIVE FUNCTION VALUES			INITIAL		31.897			31.897	
			FINAL						

S I M U L A T E D   V A L U E S

	OPTIMISATION WITH INITIAL VALUES ALL FLOODS		OPT. WITH PSP=10 ALL FLOODS		OPT. WITH PSP=10 RGF=18 ALL FLOODS		OPT. WITH PSP=10 RGF=18 FLOODS 2,3,4,5,6,8,9		OPT. WITH PSP=10 RGF=18 FLOODS 2,4,6		OPT. WITH PSP=10 RGF=18;BMSN=10 FLOODS 2,4,6	
	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
	552	0.429	536	0.411	549	0.420	374	0.305	1531	1.095	2142	1.342
	2254	2.430	2232	2.440	2257	2.485	1600	1.730	2848	3.579	3183	4.047
	3576	4.665	3612	4.754	3629	4.812	3429	3.892	3710	5.428	3723	5.479
	1338	3.556	1428	3.561	1461	3.619	1037	2.704	1839	4.225	1982	4.36
	3725	4.143	3745	4.248	3749	4.300	3629	3.473	3789	4.862	3796	4.964
	843	0.617	653	0.490	704	0.525	328	0.274	1322	0.921	1448	0.999
	833	0.557	737	0.502	781	0.527	251	0.210	1322	0.825	1442	0.885
	2181	1.732	2668	2.180	2272	1.778	1447	1.147	2840	2.442	2344	1.899
	2542	1.612	2619	1.619	2664	1.722	2219	1.316	2883	2.019	2943	2.074
	957	0.873	1058	0.922	1083	0.943	752	0.648	1377	1.197	1443	1.251
	465	0.376	496	0.398	508	0.407	350	0.297	688	0.526	744	0.563
	2217	1.312	2111	1.245	2172	1.282	1783	1.054	2365	1.443	2889	1.948
PSP		13.890		10.000		10.000		10.000		10.000		10.000
KSAT		0.205		0.245		0.240		0.342		0.188		0.180
RGF		14.680		16.010		18.000		18.000		18.000		18.000
BMSN		7.730		6.350		7.316		8.970		6.299		10.000
OBJECTIVE FUNCTION VALUES INITIAL		31.897		26.910		25.029		12.730		3.934		2.726
OBJECTIVE FUNCTION VALUES FINAL		6.707		7.500		7.260		5.550		1.465		0.049

TPEAK = TIME TO PEAK IN HOURS   PEAK = PEAK DISCHARGE IN CUSECS
EFF.RAIN = EFFECTIVE RAINFALL   IN INCHES

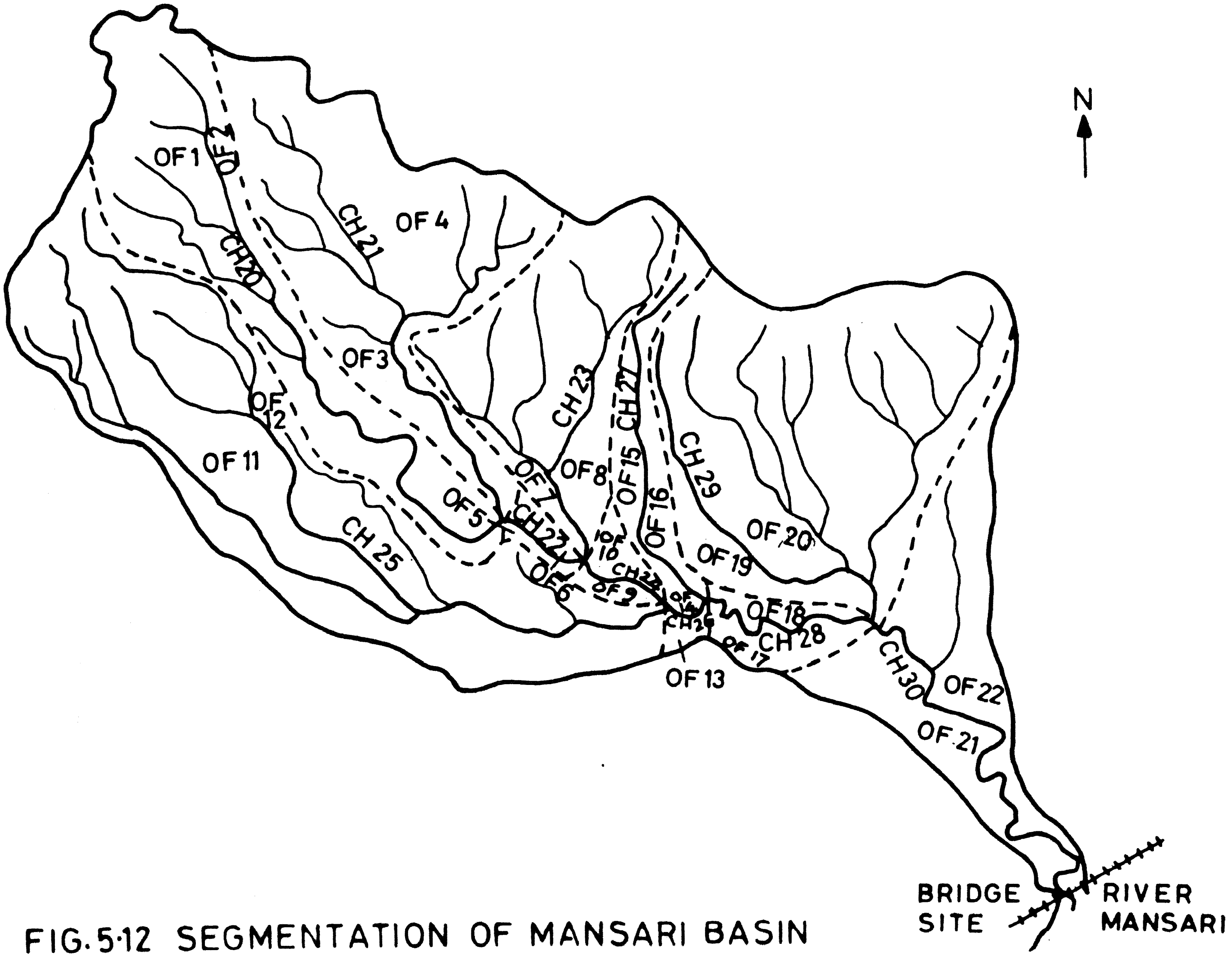


FIG. 5·12 SEGMENTATION OF MANSARI BASIN

TABLE 5.11 RESULTS OF ANALYSIS FOR MANSARI BASIN

FLOOD NUMBER	OBSERVED VALUES			NO OPTIMISATION		
	PEAK	EFF.RAIN	TPEAK	PEAK	EFF.RAIN	TPEAK
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	368	0.025	13	47	0.025	5
2	913	0.582	25	8601	0.922	16
3	523	0.242	22	5281	0.376	15
4	1414	1.102	25	6503	1.408	17
5	1583	0.706	21	9851	1.127	15

S I M U L A T E D V A L U E S

	OPTIMISATION WITH INITIAL VALUES ALL FLOODS		OPT. WITH PSP=10 ALL FLOODS		OPT. WITH PSP=10 RGF=18 ALL FLOODS		OPT. WITH PSP=10 RGF=18 FLOODS 2,3,4,5		OPT. WITH PSP=10 RGF=18;BMSN=10 FLOODS 2,3,4,5	
	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	31	0.026	29	0.024	27	0.023	13	0.015	0.07	0.0002
	2704	0.922	2397	0.826	2185	0.766	1717	0.616	1611	0.622
	794	0.376	739	0.409	799	0.44	478	0.256	433	0.243
	3303	1.408	3055	1.323	2314	1.066	2131	0.938	2117	0.976
	3082	1.127	2788	1.026	2016	0.78	1928	0.735	1950	0.786
PSP		6.000		10.000		10.000		10.000		10.000
KSAT		0.127		0.095		0.092		0.128		0.133
RGF		9.023		6.970		18.000		18.000		18.000
BMSN		4.511		4.250		1.122		3.522		10.000
OBJECTIVE FUNCTION VALUES INITIAL		4.517		4.749		6.060		0.396		0.230
OBJECTIVE FUNCTION VALUES FINAL		2.779		2.765		2.797		0.034		0.031

TPEAK = TIME TO PEAK IN HOURS PEAK = PEAK DISCHARGE IN CUSECS
EFF.RAIN = EFFECTIVE RAINFALL IN INCHES

optimisation using the initial set of parameter values also showed that the simulated peaks were very high. The objective function decreased slightly from 4.517 to 2.779, showing a large deviation between observed and simulated hydrographs during the flood events. Simulation with a number of combination of parameters also indicated high peaks. A comparison of times to peak indicated the need for additional storage elements in the river and this is considered in Sec.5.4.3.

5.4.3 Simulation with storage

The three basins, Khaga, Dharsi and Mansari, are larger in catchment area in comparison to the other basins. The necessity of introducing a storage component in the watershed model was indicated in the earlier phase of the study (Sec.5.4.2). Linear storage elements with storage discharge relationship $S = KQ$ with storage coefficient K can be used in the model at appropriate locations. For simplicity, a single lumped linear reservoir at the gauging site was adopted in this study. Trial values of storage coefficients around the indicated values were tested without optimisation and with optimisation to decide on the storage coefficient K and appropriate parameters of the distributed parameter model. For example, results of analysis for Dharsi basin are shown in Table 5.12. This indicates a storage coefficient of $K = 6$ hours and the corresponding set of parameters. Similar studies for Khaga and Mansari showed a delay time of 2 hours and 9 hours respectively.

5.4.4 Different infiltration conditions

For four basins, viz., Sankhini, Mujnai, Khaga and Dharsi, there are two

TABLE 5.12 RESULTS OF ANALYSIS WITH STORAGE FOR DHARSI BASIN

	OBSERVED			SIMULATED (WITHOUT OPTIMISATION)						
FLOOD NUMBER	PEAK	EFF.RAIN	TPEAK	PEAK K = 1 HOUR	EFF.RAIN	TPEAK	PEAK K = 3 HOURS	TPEAK	PEAK K = 6 HOURS	TPEAK
1	2	3	4	5	6	7	8	9	10	11
1	743	0.499	13	4409	2.814	7	3821	13	3421	15
2	3037	3.548	35	5187	7.414	8	4406	9	3766	34
3	2522	3.082	39	5998	8.047	16	4469	33	4270	36
4	1379	1.291	15	4568	7.601	9	3864	15	3613	16
5	1245	0.781	13	5857	7.047	8	4975	8	3841	11
6	1713	1.178	17	4506	2.653	13	3979	16	3398	17
7	880	0.717	10	4041	1.941	6	3597	10	2930	11
8	1773	1.059	14	5463	7.508	31	4253	32	3810	33
9	1658	1.001	11	5280	4.378	5	3837	20	3692	21
10	582	0.434	9	4665	4.449	6	3896	9	3621	10
11	688	0.532	16	4453	2.357	8	3852	11	3201	15
12	702	0.484	14	4862	3.717	10	3839	22	3640	22

	OBSERVED			SIMULATED (WITH OPTIMISATION)						
FLOOD NUMBER	PEAK	EFF.RAIN	TPEAK	PEAK K = 1 HOUR	EFF.RAIN	TPEAK	PEAK K = 3 HOURS	TPEAK	PEAK K = 6 HOURS	TPEAK
1	2	3	4	5	6	7	8	9	10	11
1	743	0.499	13	1044	0.424	9	764	9	552	12
2	3037	3.548	35	3803	2.43	32	3092	33	2254	34
3	2522	3.082	39	4677	4.605	15	3938	20	3576	27
4	1379	1.291	15	2242	3.556	9	1808	11	1338	11
5	1245	0.781	13	5210	4.143	8	4127	9	3725	21
6	1713	1.178	17	1589	0.617	15	1177	17	843	18
7	880	0.717	10	1768	0.557	7	1212	8	833	9
8	1773	1.059	14	3855	1.732	32	3069	33	2181	33
9	1658	1.001	11	3839	1.612	10	3355	11	2542	11
10	582	0.434	9	2001	0.873	8	1371	9	957	10
11	688	0.532	16	841	0.376	12	638	13	465	14
12	702	0.484	14	3841	1.312	13	3044	13	2217	14

	PSP	13.893
	KSAT	0.205
	RGF	14.68
	BMSN	7.727
OBJECTIVE	INITIAL	31.93
FUNCTION VALUE	FINAL	6.711

PEAK = PEAK DISCHARGE IN CUSECS
EFF.RAIN = EFFECTIVE RAINFALL IN INCHES
TPEAK = TIME TO PEAK IN HOURS

distinct groups of floods due to differring infiltration conditions. For example, for Sankhini basin (Table 5.7), flood events 2 and 4 pertain to a low infiltration condition while flood events 1 and 3 correspond to a high infiltration condition. Since the present study is concerned with estimation of design flood, the parameter estimates corresponding to those of lower infiltration conditions are considered to be appropriate for the purpose of the study. Hence optimising the two parameters KSAT and BMSN with PSP = 10.00 and RGF = 18.00 for floods 2 and 4 of Sankhini basin, the values of these parameters are obtained as KSAT = 0.139; and BMSN = 10.395. For the floods under higher infiltration conditions, these values are 0.277 and 8.045 respectively. These are shown in columns 12 to 15. For the lower infiltration conditions, the simulated values of peak and the effective rainfall are found to be slightly low for flood event 2 while it is high for the flood event 4. The values for the other floods are farther away, being very high in the case of floods 1 and 3 and very low in the case of flood 5. The objective function value reduces form 0.662 to 0.192 in this case.

Similarly for Mujnai basin, (Table5.8), flood events 1,2,3 and 9 (Fig.5.12a) pertain to a low infiltration condition while flood events 6,7, and 8 correspond to a high infiltration condition. Flood events 5 and 10 have comparable values and flood event 4 corresponds to a very wet condition. For further analysis flood events 5,6,7,8 and 10 (Fig.5.12b) are only considered since for design flood estimation, parameters corresponding to wet infiltration conditions are considered to be appropriate.

Similarly for Dharsi basin (Table 5.10), flood events 2, 3, 4, 5, 6, 8, and 9 are the major floods. Keeping the values of PSP = 10.00 and RGF = 18.00, the simulation run for these floods yield values of KSAT = 0.342 and BMSN = 8.97.

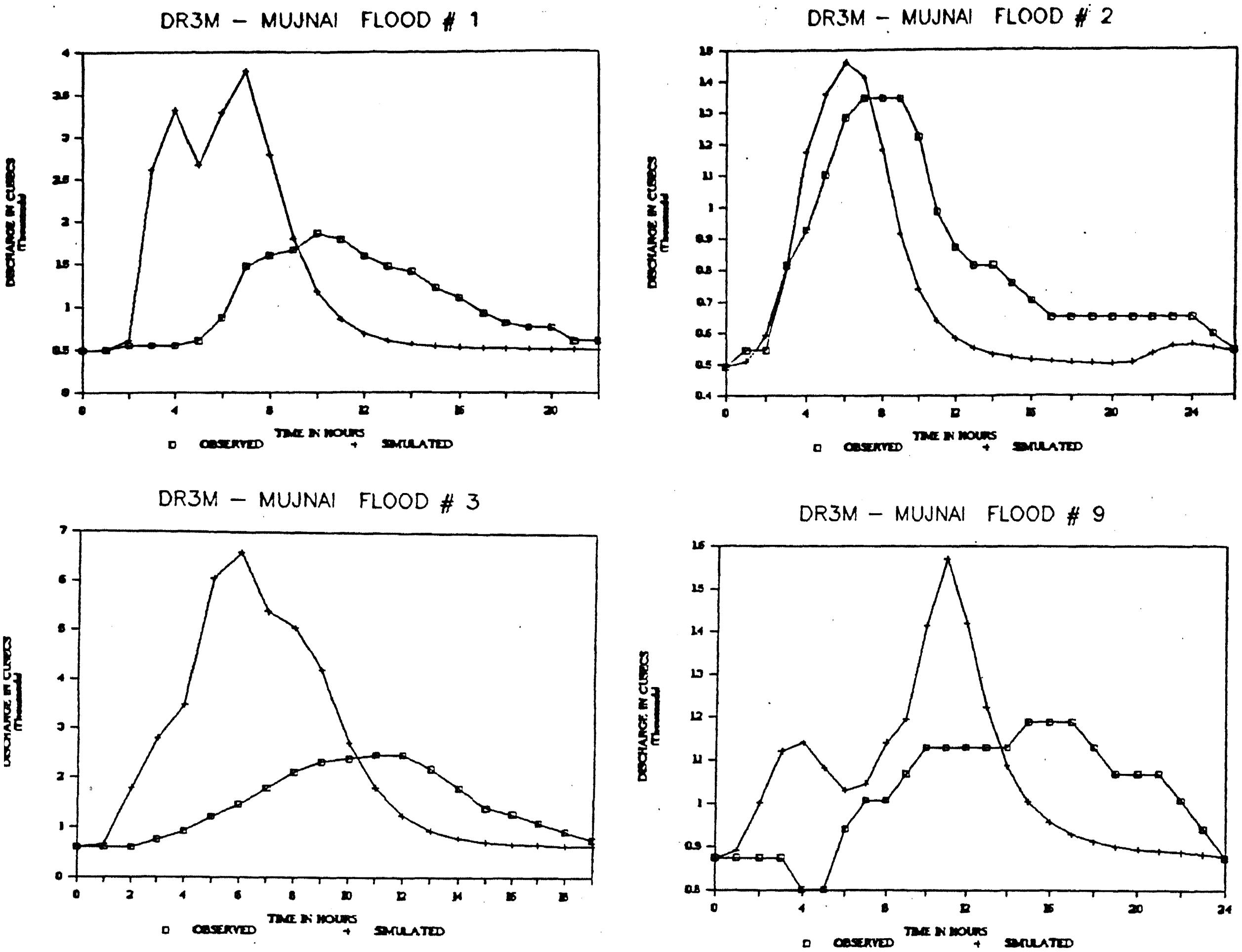


FIG. 5.12 a. SIMULATED HYDROGRAPHS OF MUJNAI BASIN (Dry initial condition)

This value of KSAT indicates that there is a dominance of some floods and higher infiltration conditions. Flood events 2, 4, and 6 correspond to lower infiltration conditions during actual storm as is evident from the lower values of simulated peaks and effective rainfall and correspondingly the higher values of historical flood peaks and effective rainfall. Simulating these three floods with the constant values of PSP and RGF, yield values of KSAT = 0.188 and BMSN = 6.299. The objective function value for this condition has an initial value of 3.934 and a final value of 1.465.

For Khaga basin (Table 5.9), when simulation is carried out for all the floods,

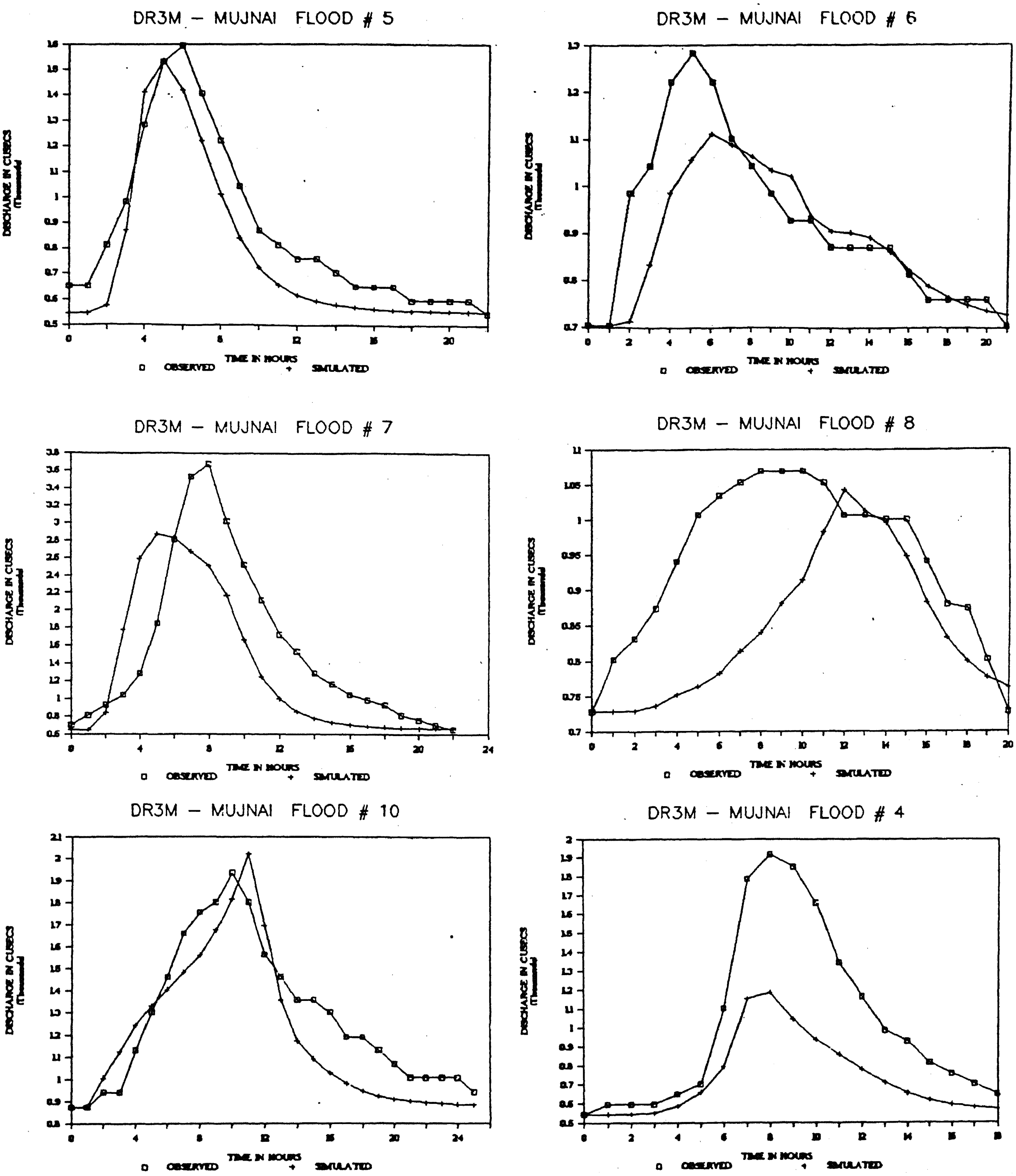


FIG.5.12 b. SIMULATED HYDROGRAPHS FOR MUJNAI BASIN

Floods 5,6,7,8 and 10 --> Wet initial condition; Flood 4 --> Very wet initial condition

Table 5.13 Model parameters after initial simulation

Constant parameters : EVC = 0.70; RR = 0.90; DRN = 0.50

Basin	Area Km^2	PSP inches	KSAT inches per hr	RGF	BMSN inches	Objective Function Initial	Objective Function Final	K Hours
Batjhora	7.33	7.586	0.112	16.46	1.89	2.380	1.126	
Sankhini	7.76	3.052	0.366	17.40	12.59	4.850	0.632	
Mujnai	38.40	9.909	0.198	19.42	1.025	12.081	1.563	
Talma	42.10	8.179	0.167	16.63	7.731	8.699	5.762	
Khaga	66.00	3.147	0.369	18.29	6.310	5.255	3.265	2.0
Dharsi	92.40	13.890	0.205	14.68	7.730	31.897	6.707	6.0
Mansari	213.00	6.00	0.127	9.02	4.511	4.516	2.779	9.0

the other three parameters are optimised, the objective function value at the end of the optimisation run is 5.999. This differs only slightly from the earlier value of 5.762. It is also found that by changing the value of PSP to 10.00, there is only marginal change in the values of flood peaks and the effective rainfalls and in a few cases there is an improvement. Similarly, comparison of columns 8 and 9 with columns 10 and 11 shows that when PSP = 10, changing RGF to 18.00 produces no significant change in the final objective function value (6.011) or in the peak and the effective rainfall values. Similar studies for all the basins indicate that the two parameters PSP and RGF can be taken as constant within the observed range for all the basins with values of 10.00 and 18.00 respectively. (Table 5.13). The results of the simulation run with optimisation with the values of PSP = 10.00 and RGF = 18.00 for the model are shown in Tables 5.5 to 5.13 and are summarised in the Table 5.14.

Table 5.14 Model Parameters Values with PSP = 10; and RGF = 18.00

Constant Parameters: PSP = 10.00; RGF = 18.00; EVC=0.70; RR=0.90; DRN=0.50				
Basin	KSAT	BMSN	Objective Function	
			Initial	Final
Batjhora	0.096	1.180	0.30	0.22
Sankhini	0.171	8.740	4.12	1.29
Mujnai	0.103	5.170	9.36	8.49
Talma	0.163	8.198	7.53	6.01
Khaga	0.165	6.000	4.47	3.73
Dharsi	0.240	7.320	25.03	7.30
Mansari	0.092	1.122	6.06	2.80

The corresponding values of peak and the effective rainfall to this run are also shown in Tables 5.5 through 5.11.

It is seen from Table 5.14 that the BMSN values range from 1.12 to 8.74 inches. A number of studies with varying values of BMSN were done and it was observed that the model is not very sensitive to this parameter. A value of 10 inches for BMSN corresponding to an upper limit (Table 5.13) is adopted. Thus KSAT is found to be the only sensitive parameter of the model for simulating floods in the region. Simulation is carried out to optimise this parameter (KSAT) for each of the basins and the results are presented in the last two columns of Tables 5.5 through 5.11. It may be seen that on the basis of peak and effective rainfall of each of the floods and the value of the objective function the results are comparable to those of the respective previous columns. Accordingly constant parameter values of PSP=10.00 inches; RGF=18.00; and BMSN=10.00 inches are

indicated as regional parameters values and they are adopted in the study. KSAT, however, was one of the parameters to which the results of simulation were sensitive. The values of KSAT for various basins, assuming the other parameters as constant are shown in Table 5.15 and Fig. 5.14

Table 5.15 Variation of KSAT in the region among the basins

Constant Parameters: PSP = 10.00; RGF = 18.00; BMSN = 10.00; EVC = 0.70; RR = 0.90;DRN = 0.50

Basin	KSAT inches/hour	Objective Function Initial	Final
Batjhora	0.102	1.226	1.215
Sankhini	0.135	1.629	0.201
Mujnai	0.173	1.450	0.505
Talma	0.182	0.343	0.303
Khaga	0.123 *	5.860	5.820
Dharsi	0.180	2.726	0.449
Mansari	0.133 *	0.230	0.031

*** Values of the parameter inconsistent.**

It is observed from the Table and Fig.5.14 that the values of KSAT for the smaller basins, viz., Batjhora and Sankhini are very low, apparently due to the presence of impervious areas which are not considered in the present study so far. The values of KSAT for the other three basins considered are 0.173 inches/hour for Mujnai, 0.182 inches/hour for Talma and 0.180 inches/hour for Dharsi. The average value of KSAT for these three basins is 0.178 inches/hour (4.53 mm/hour) and this compares well with 5.00 mm/hour value of infiltration index used in Sec. 4.6

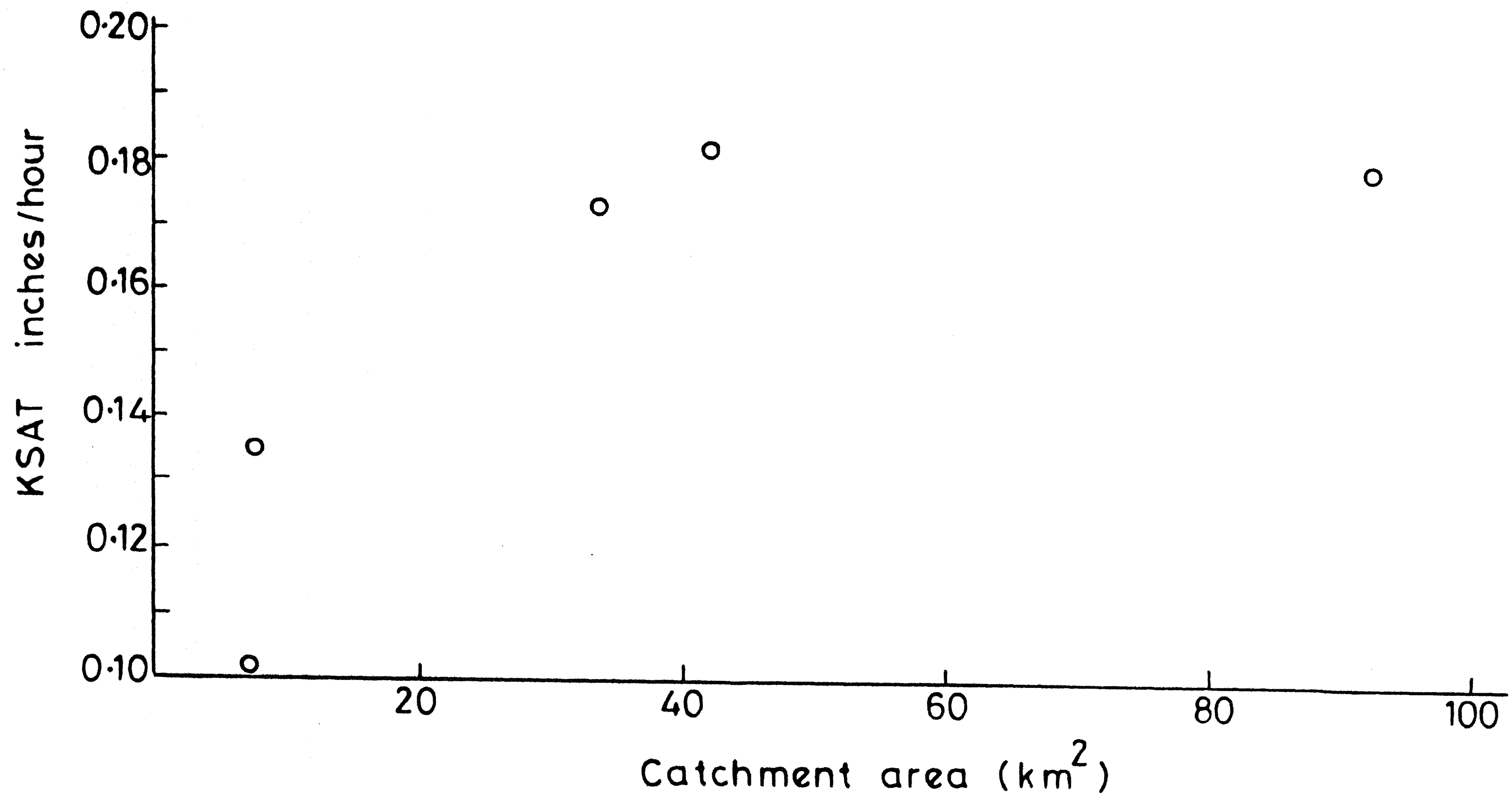


FIG.5·13 VARIATION OF KSAT WITH CATCHMENT AREA

and agrees well with the silty loam or sandy clay loam soil present in the region. It was decided to use this value of KSAT for the smaller basins with directly contributing impervious percentage of catchment area as a parameter. Simulation was carried out with a value of KSAT = 0.178 inches/hour and a few trial runs showed that 20% of the area as impervious for Batjhora and 10% for Sankhini, the results of the simulation are comparable to the earlier runs. It was then decided to use this value of KSAT and simulate floods in all the basins and the results are shown in Table 5.16. These results seem to be realistic and hence are accepted in the study.

For the region under study, the following parameters are found to be regional parameters;

PSP	10.000 inches
KSAT	0.178 inches/hour
RGF	18.000
BMSN	10.000 inches
EVC	0.700
RR	0.900
DRN	0.50

5.6 Application of DR3M to Kurti Basin

Kurti basin was studied in Subsec. 4.5.1 and rejected for further study because of suspected data errors. Results of simulation of floods for this basin using DR3M with the model parameters for the region with historical data for rainfall are shown in Table 5.17. They show wide variation between the observed and simulated flood runoffs and peaks. The floods and peaks are much larger in the

TABLE 5.16 RESULTS OF ANALYSIS FOR ALL THE BASINS USING THE REGIONAL PARAMETERS

	BATJHORA				SANKHINI				MUJNAI			
	OBSERVED		SIMULATED		OBSERVED		SIMULATED		OBSERVED		SIMULATED	
	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
	1	2	3	4	5	6	7	8	9	10	11	12
	160	0.356	123	0.251	338	1.666	492	2.924	1359	1.313	3659	2.465
	399	1.322	1708	1.503	263	1.869	171	1.318	851	0.932	1418	1.220
	310	0.848	303	0.727	279	2.529	506	4.542	1851	1.778	6102	4.135
	233	0.590	345	0.696	168	1.149	189	1.647	1372	0.938	699	0.565
	310	1.399	974	1.064	132	0.642	83	0.564	1048	0.775	1094	0.833
	813	3.779	1057	2.051					579	0.490	453	0.638
	813	2.470	650	1.688					3028	2.181	2361	2.556
	163	0.567	548	1.144					341	0.537	346	0.367
	194	0.385	486	0.705					387	0.658	779	0.694
									1062	1.078	1246	1.207
ERVIOUS 'A PERCENTAGE			20				10					

TALMA				KHAGA				DHARSI			
OBSERVED		SIMULATED		OBSERVED		SIMULATED		OBSERVED		SIMULATED	
PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN	PEAK	EFF.RAIN
13	14	15	16	17	18	19	20	21	22	23	24
179	0.147	121	0.263	13365	5.626	5450	2.394	743	0.499	2116	1.329
396	0.732	82	0.199	5371	2.106	5645	2.635	3037	3.548	3168	4.022
739	1.548	202	0.441	1466	0.615	750	0.342	2522	3.082	3122	5.554
4704	8.401	5004	8.374	1999	1.369	4783	3.318	1379	1.291	1979	4.354
251	0.418	56	0.145	1319	0.840	5483	3.587	1245	0.781	3796	4.959
501	0.710	721	1.369					1713	1.178	1441	0.955
1213	2.359	1147	2.499					880	0.717	1436	0.882
625	0.357	436	0.735					1773	1.059	2333	1.894
3093	5.997	3936	5.680					1658	1.001	2942	2.071
3523	9.741	2846	7.006					582	0.434	1441	1.248
2703	3.126	2875	4.820					688	0.532	742	0.562
598	0.650	517	0.967					702	0.484	2886	1.940

MANSARI			
OBSERVED		SIMULATED	
PEAK	EFF.RAIN	PEAK	EFF.RAIN
25	26	27	28
368	0.025	51	0.038
913	0.582	1932	0.719
523	0.242	586	0.395
1414	1.102	2606	1.196
1583	0.706	1492	0.899

PARAMETERS:

PSP	10.000	INCHES
KSAT	0.178	INCHES/HOUR
RGF	18.000	
BMSN	10.000	INCHES
EVC	0.700	
RR	0.900	
DRN	0.500	

PEAK = PEAK DISCHARGE IN CUSECS
EFF.RAIN = EFFECTIVE RAINFALLIN INCHES

case of flood events 1 to 3 and less in other cases. Perhaps as in the case of Sankhini, Dharsi and Khaga basins, the results indicate different infiltration conditions, viz., a wet infiltration condition for floods 1 to 3 resulting in high floods and dry infiltration conditions for floods 4 and 5. The results confirm the earlier suspicion of data errors.

Table 5.17 Results of simulation of floods for Kurti basin

Flood No.	Observed			Simulated		
	Actual Peak		Effective Rainfall Inches	Peak		Effective Rainfall Inches
	Ft^3/sec	M^3/sec		Ft^3/sec	M^3/sec	
1	1067	30.22	2.440	2601	73.77	7.491
2	1067	30.22	1.879	1912	54.15	4.270
3	2244	63.55	5.946	4099	116.1	14.650
4	1476	41.80	2.608	745	21.09	1.400
5	1393	39.45	2.855	661	18.72	1.700

5.6.1 Estimating design flood peak for Kurti basin

To estimate the design flood for the Kurti basin, the antecedent conditons for design storm are to be simulated in terms of specified wet initial soil moisture conditions. In case storm precipitation data for the given basin were not available, it is possible to use precipitation data from an adjacent raingauge station or basin in the simulation. The design storm could have been introduced when the simulated discharge corresponds to design baseflow (Fig.4.11). However, since precipitation data are available for the basin, design storm was introduced immediately following the

day of the highest simulated flood, viz., flood event 3 and the simulation was carried out using the design storm of duration 2 hours which is the time of concentration for the basin, and a 12 hour design storm. Since the simulated flood peak for flood 3 is of the order of 116.1 m^3/sec, much larger than the design flood peak, to consider other initial wet conditions the design storm is introduced separately also immediately following the day of the second highest simulated flood (Q_{peak} = 73.77 m^3/sec), viz., flood event 1 and the results are also summarised in Table 5.18.

Table 5.18 Results of design flood estimation

Design storm duration hours	Peak discharge value Ft³/sec	M³/sec	Baseflow Ft³/sec	M³/sec	Peak discharge with baseflow of 25.44 M³/sec
(1)	(2)	(3)	(4)	(5)	(6)
Case (i) Design storm follows highest simulated storm flood event					
2	3037	86.01	1259	35.63	75.82
12	3157	89.41	1259	35.63	79.22
Case (i) Design storm follows second highest simulated storm flood event					
2	2722	77.09	835	23.65	78.88
12	3030	85.81	835	23.65	87.60

Since the baseflows are different for the two cases studied, as an approximation, a correction for difference in baseflow is applied to the simulated peak discharges and the corrected peak values are shown in column 6. The results indicate that the design peak discharges are comparable to those of Sec.4.7. Furthermore a large initial baseflow perhaps may lead to a smaller peak discharge. A 12 hour design storm hyetograph seems to indicate a realistic design peak in both cases.

Accordingly the following steps for estimation of design flood hydrograph for an ungauged basin is suggested.

i) From available physiographic and hydrometeorological data regional parameters for the model are estimated including their variations, if any, in space or as a function of area.

ii) Physiographic data for the basin are collected and adapted in the model.

iii) Using precipitation data for the basin, if avialable, or else precipitation data for a nearby station or basin, the watershed is simulated to identify the larger peaks. Data for a couple of years is considered to be adequate.

iv) Using IMD data, a 12 hour design storm for the basin is estimated and is introduced on the day when the simulated discharge is approximately equal to the design baseflow for the basin. Alternatively the design storm may be introduced the day next to that of the high peaks.

v) By simulation the design flood is obtained. In case the baseflow is different from the design baseflow, the design flood hydrograph may be may be appropriately corrected.

5.7 Conclusions

The study indicates that DR3M with constant parameters for the region performs satisfactorily for the small basins in terms of prediction of peaks and effective rainfall. From consideration of parsimony of parameters, a simple constant parameter model is considered to be realistic representation of small basins in the region.

The conclusions have been reached in a heuristic manner. The UH approach could be used in case adequate rainfall-runoff data are available or if a regional relationship on the basis of UH has already been obtained. But since the relationships are essentially derived based on basin area or similar parameters empirically, they may be subject to large errors and are not recommended.

Topography, channel crosssections and soil data may be easily availble for field organisations and can be used with physically based regionally derived parameter estimates in a distributed parameter model like DR3M to derive design flood. Accordingly a distributed parameter model is recommended for flood estimation in small catchments except when at least some rainfall - runoff data are available for the basin.

CHAPTER 6

SUMMARY, CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

6.1 Summary:

Estimates of flood are required for the design and economic appraisal of a variety of engineering works such as culverts, spillways etc. Limited or non-availability of data is a major constraint in flood estimation for watersheds.

Several approaches are available for estimation of floods. When adequate and appropriate data are available for the basin, frequency analysis may be applied or else unit hydrograph (UH) and other rainfall-runoff procedures may be applied to model the basin system. When this is combined with design rainfall, the design flood may be derived. Data available for small watersheds are generally inadequate for these approaches. Regional flood frequency analysis is generally not valid for small watersheds because of data limitations. Then other empirical approaches for regionalisation are to be used to model either the variation of flood peaks or rainfall-runoff relationships.

From available data regression relationships are generally derived to relate peak discharge to hydrometeorological features of the storm and physiographic features of the watershed. Even when adequate data are available for establishing such relationships, they are limited to the regions for which such relationships have been derived. Since generally adequate data are available for estimating design storms, it seems worthwhile to derive relationships between storm rainfall and streamflow, perhaps in terms of unit hydrographs. The parameters of the UH may then be related to the characteristics of the storm and the physiographic features of the watershed to yield a regional relationship for design purpose between storms

and floods.

A more realistic approach is to model the catchment runoff process according to physical principles. Data concerning topography, soil and vegetation are available from remote sensing. The physically based models can now be implemented even in a PC environment. There has also been a greater availability of technical know-how on the applicability of such models. Hence, it seems worthwhile to consider the use of physically based distributed parameter models as an alternative to regionalisation for flood estimation in small catchments.

Information regarding physiography, soils, vegetation and the drainage pattern of the catchment is used in the physically based models. The catchment is subdivided into a network of channel and overland flow segments. Using the physical characteristics of each component and appropriate hydraulic equations, the hydrologic processes are simulated using the distributed parameter model. Necessary values of the coefficients and constants are specified from physical characteristics of the watershed or obtained from calibration of the model with observed data.

For purposes of hydrometeorological studies, India has been divided into 26 subzones by the Central Water Commission (CWC), each of which is assumed to be hydrologically homogeneous. Research Designs and Standards Organisation (RDSO) of the Ministry of Railways has collected, and in collaboration with CWC and India Meteorology Department (IMD) has analysed a large amount of hydrometeorologicl data for a large number of catchments draining into various railway culverts and bridges in different subzones. This study is essentially a regional case study for flood estimation in small catchments in a part of the north Brahmaputra basin, designated as subzone 2(a), to compare particularly in a very limited data environment the two approaches for rainfall-runoff relationships, viz.,

regionalisation using UH and a physically based model. For a number of small watersheds in this region, available hydrometeorological data were very limited and inadequate and so data available were yet to be analysed and interpreted.

The objectives of the present study are:

i) To develop a regional relationship for UH parameters from available limited data for a number of basins to evaluate the validity and applicability of UH based regionalisation procedure for flood estimation.

ii) To apply a physically based model to small watersheds in the region to evaluate the validity of such a model to model the regional variation, if any, of the parameters of the model and also to judge their validity for flood estimation, and

iii) To compare these two approaches for regional flood estimation for small catchments.

The study is limited to small watersheds in the western part of the north Brahmaputra basin, for which data were available. Furthermore, the UH is represented by the Nash conceptual model and the physically based model considered in the study is the USGS Distributed Routing Rainfall Runoff Model (DR3M).

Catchment plan, crosssection at the bridge site and hourly rainfall and stage data at railway bridges for about 10 catchments in the region were available for the study. The catchment areas ranged from about 7.00 Km^2 to 215.00 Km^2. The data were available for a period ranging from two to three years. After preliminary analysis for data errors, only data of seven basins were found to be suitable for analysis.

In the UH approach for regionalisation, it is necessary to represent the UH in terms of a limited number of parameters. A conceptual model with two parameters N

and K proposed by Nash was adopted in the study. The various steps involved in the regionalisation of the parameters of this model are:

i) Estimate the UH parameters for a storm in a basin,

ii) Repeat the step for a number of storms in a basin to explain the variation, if any, of the parameters in terms of hydrometeorological characteristics of the storm, and

iii) Repeat the above two steps for a number of basins in the region so that the residual variation of parameters can be correlated to the physiographic characteristics of the basin.

Various parameters like the coefficient of runoff, the infiltration index and baseflow are found to vary from storm to storm in each basin rather erratically. Attempts made to correlate them with storm and runoff parameters were unsuccessful. The variation of the infiltration index among the storms was very large. Correlation with a number of storm flood parameters also showed wide variation. However, a general study of the individual storms indicated that a value of 5.00 mm/hour can be considered to be the general minimum value for the infiltration index for all the basins, particularly during intense storms generally met with in the design. This value corresponds to the silty loam soil or sandy clay loam soil of the region.

The UH parameters N and K of the Nash model vary from storm to storm and from basin to basin. A smaller NK generally leads to a quicker flood and a high peak. A quasilinear variation of N and K as a function of peak discharge (Q_{peak}) is assumed and for each of the basin, the regression equations relating N, K and NK with Q_{peak} is derived. Regional regression equations relating N and K corresponding to the maximum observed peak in each basin are as follows:

$N = 1.792 + 0.0088\ A$, and $K = 2.766 + 0.016A$

where A is in Km^2, K is in hours and N is dimensionless.

The baseflow varies from storm to storm and generally increases with the basin area. Baseflow/Km^2 for all critical storms are plotted as a function of basin area and an enveloping curve is drawn. Since the higher values of baseflow lead to larger floods, the enveloping curve for baseflow/Km^2 is recommended for use with the design flood.

A design flood estimated using the regional relationships for a small watershed in the region is derived and is compared with other approaches generally used in India. The results are found to be satisfactory.

Many distributed parameter models are available for simulating rural and hilly catchment runoff. The DR3M has been tested for some rural and hilly basins in India with satisfactory results and hence this model was adopted for the study. A drainage basin is represented in this model as a set of channel and overland flow segments in such a way that the essential basin geometry and physiography are taken into account for runoff computation. The model has four components - a soil moisture or water balance component, the infiltration or rainfall excess component, a routing component and an optimisation component. The model has seven physically based parameters - four soil moisture accounting parameters, viz., (i) DRN, a constant drainage rate for redistribution of soil moisture in inches/day, (ii) EVC, a pan coefficient for converting measured pan evaporation to potential evapotranspiration; (iii) RR, the proportion of daily rainfall that infiltrates into the soil for period of simulation excluding unit rainfall days and (iv) BMSN, the soil-moisture storage at field capacity in inches, and three infiltration parameters viz., (i) PSP, the suction at wetted front for soil moisture at field capacity in inches of

pressure, (ii) KSAT, the effective saturated value of the hydraulic conductivity in inches/hour and (iii) RGF, the ratio of suction at wetted front for soil moisture at wilting point to that at field capacity. These parameters are fitted to the model for a basin by Rosenbrock's algorithm and the iterative optimisation among basins is done heuristically. The results of the study indicated that a constant parameter model for the small catchments in the region is realistic. The model parameters agree with the watershed characteristics.

It is realised from the study that the directly contributing impervious area percentage to be given as input data from the characteristics of the watershed is very important in estimating the flood peak. Such data were not available for the region. They were parameterised and estimated by simulation and compared with general topography and basin area and are considered to be satisfactory. However, with the use of remote sensed data (not available for the study but generally available for government and other field organisations), it will be possible to use this information for realistic modelling of small watersheds. The design flood estimated for a small watershed using this model compared favourably with other estimates.

6.2 Conclusions:

1. For estimation of design flood in a small basin, since design rainfall can be estimated separately by standard procedures, it seems preferable to use a regional rainfall-runoff relationship with design storm rather than using a purely empirical approach.
2. The UH parameters are not generally constant. Hence it is necessary to adopt a quasilinear approach and criteria for estimation and regionalisation of design

parameters. Regional UH approaches involve a large amount of empiricism. When sufficiently large amount of data are available they may be used for regionalisation.

3. For the design flood for the region under study, a constant infiltration parameter during the design storm, a baseflow related to the basin area and a regional quasilinear relationship for the UH parameters were found to be satisfactory.

4. DR3M is a relatively simple distributed parameter model which can be used for simulation of runoff in small watersheds.

5. Impervious area is a very important parameter in the DR3M which has to be estimated with care.

6. Even in a very limited data environment, DR3M has been found to work satisfactorily.

7. DR3M may be used to realistically estimate the design flood, particularly when data are very limited, since field data concerning topography, soil and vegetation may be easily collected by remote sensing and other means or by modelling and regionalisation.

8. Because of easy availability of, and competence with computers and softwares in field organisations, it seems preferable to adopt a distributed parameter model with relevant field data rather than empirical, statistical relationships of questionable validity.

9. Distributed parameter models, may, eventually replace empirical regression models in hydrology not only for large watersheds but also for small watersheds.

6.3 Suggestions for Further Study

Based on the results of the study, the following suggestions are made for further study:

1. The study used limited data of questionable reliability. It seems worthwhile to use reliable data of sufficiently long length.
2. Correlation and regionalisation of model parameters with physiographic characteristics of the basin need further study.
3. Comparison of results with other distributed parameter models may identify an appropriate model for use in the case of small watersheds.
4. It is also necessary to test such models in a variety of regions to identify variation of parameters in space and as a function of area. The variation, if any, may be used for defining homogeneity of a region and identification of hydrologically homogeneous regions.

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